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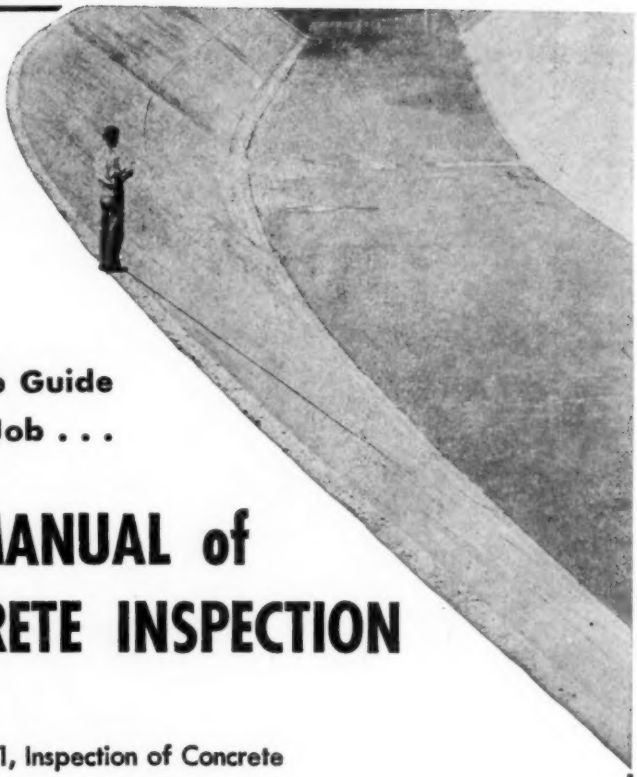

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Four recently adopted ACI standards will be published in their entirety in the July issue of the JOURNAL:

"Test Procedure to Determine Relative Bond Value of Reinforcing Bars (ACI 208-58)"

"Recommended Practice for Design of Concrete Pavements (ACI 325-58)"

"Specifications for Concrete Pavements and Concrete Bases (ACI 617-58)"

"Minimum Standard Requirements for Precast Concrete Floor and Roof Units (ACI 711-58)"

●

Part I of a series of four papers dealing with "Origin, Evolution, and Effects of the Air Voids System in Concrete," by RICHARD C. MIELENZ, VLADIMIR E. WOLKODOFF, JAMES E. BACKSTROM, and HARRY L. FLACK is scheduled for next month. Parts II, III, and IV will be published in consecutive issues of the JOURNAL.

●

"Load Test on Flat Slab Floor with Embedded Steel Grillage Caps" was prepared by DONALD D. MEISEL, CYRIL D. JENSEN, and WALTER H. WHEELER.

●

"Heavy-Media Processing of Gravels in New Brunswick" by I. D. MACKENZIE describes procedure for removal of low specific gravity materials from aggregate to produce concrete with satisfactory resistance to freezing and thawing.

●

General design data for four prestressed concrete highway bridges and two prestressed concrete railroad bridges are included in "Large Scale Test of 120-Ft Span Precast, Prestressed Bridge Girder" by FAZLUR R. KHAN and ANDREW J. BROWN. Bridges were built at the new U. S. Air Force Academy site in Colorado.

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Title No. 54-59

Long-Time Study of Cement Performance in Concrete

Chapter 11—Report on the Condition of Three Test Pavements After 15 Years of Service

By FRANK H. JACKSON†

SYNOPSIS

This report discusses the present condition of three test pavements built about 15 years ago. One pavement is in western New York in a region subject to severe natural weathering, one in central Missouri where exposure conditions are moderately severe, and one in western South Carolina where mild weather prevails. Twenty-seven cements, differing widely in their chemical and physical properties, were used in these pavements.

After 15 years service the New York pavement exhibits surface scaling in varying amounts on almost all sections containing the non-air-entraining cements. However, except for the Type IV and Type V cements, there is no indication that any one non-air-entraining cement or type of non-air-entraining cement is more resistant to scaling than another. Type IV and Type V cements show greater average resistance to scaling than the other non-air-entraining types. All sections containing air-entraining cements are still completely free from surface scale.

Aside from some light scale or surface wear on the South Carolina project and some D-cracking on the Missouri road, neither of which can be associated in any way with a particular cement or cement type, all of the cements have performed equally well on both projects. Under the conditions prevailing on these projects, variations in the chemical composition and fineness of the cement, within the limits represented by this study, appear to be without significance insofar as resistance to freezing and thawing is concerned.

INTRODUCTION

About 15 years ago, the Portland Cement Association, in cooperation with a number of federal, state, and private agencies, initiated an extensive field program to study cement performance in concrete. This investigation was the outgrowth of a suggestion made by P. H. Bates, then chief of the division of clay and silicate products at the National Bureau of Standards and also chairman of the American Society for Testing Materials Committee C-1 on cement. It was at about this time that ASTM adopted the five-type specification for portland cement, thus breaking away from the "single standard" philosophy which had dominated the committee's thinking for many years.

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†Member American Concrete Institute, Consulting Engineer, Chevy Chase, Md.

During the decade just preceding this ASTM action, there were many discussions regarding the desirability of setting up requirements to cover portland cement to meet specific needs, such as early strength, low heat of hardening, and sulfate resistance. Also, many user organizations, not satisfied with ASTM's single standard, were issuing their own specifications. These included the highway departments of New York and certain other states whose specifications included special requirements covering compound composition; the specifications for "low-heat" and "moderate-heat" cements of the U. S. Bureau of Reclamation and the Federal Specification Board; the "Type B" cement specifications of the Tennessee Valley Authority; and, finally, the elaborate specifications covering manufacturing controls issued by the New York Board of Water Supply under guidance of the late Thaddeus Merriman.

One of the most controversial questions involved in these discussions concerned the need for a so-called Type II cement and, if such a need existed, whether it should be considered as an improved "general purpose" cement or merely defined as a moderate-heat cement. Those in favor of the first position cited numerous laboratory data, including extensive tests by the U. S. Bureau of Reclamation which showed that resistance to frost action was improved by limiting the quantity of potential tricalcium aluminate (C_3A) in the cement. Those in favor of the second position pointed to the excellent service records of many concrete structures containing cements high in C_3A. The final decision of the ASTM was to delete reference to improved durability in the specifications for Type II cement and to consider it strictly as a cement with moderate heat of hydration and moderate sulfate resistance.

Another problem giving much concern 15 years ago was surface deterioration. Pavements in many of the northern states where chemicals were being used for ice removal were beginning to scale badly and, in some cases, to develop D-lines* (cracks) along joints and cracks which, it was feared, might be the prelude to early disintegration. Highway engineers were greatly concerned and they scrutinized cement as one possible source of trouble. At that time the great improvement in the durability of concrete made possible by the use of air entrainment was just beginning to be realized, and many engineers were not convinced—incidentally some of them are still not convinced—that the answer to the durability problem lay in the proper application of this principle rather than in restrictions in the chemical composition of the cement.

The foregoing constitute some of the reasons that prompted Mr. Bates to suggest to Frank Sheets, then president of the Portland Cement Association, that the industry sponsor a program to study the performance of cement in job-size concrete structures of various types and so located as to be exposed to a wide variety of climatic conditions. The idea met with ready

*In a concrete pavement, deterioration due to weathering is usually evidenced first by the appearance of fine, more or less closely spaced parallel surface cracks along the joints and structural cracks and sometimes along the free edges of the pavement slab. These cracks are frequently filled with a dark colored deposit—principally calcium carbonate—whence the name "D-cracks" or "deposit-cracks."

acceptance, and the Portland Cement Association appropriated $350,000 to carry out the work. An advisory committee was appointed, consisting of eight members representing consumer groups and four members representing the cement industry, with Mr. Bates as chairman. Several changes in the personnel of this committee have since taken place, and the committee is now under the chairmanship of R. F. Blanks. This committee has had complete responsibility for the conduct of the investigation from the start. It developed the original program and has reviewed and approved all progress reports thus far published.

Long-Time Study cements

Twenty-seven different portland cements, varying in physical and chemical properties through the entire range permitted under the then current specifications of the American Society of Testing Materials (C 150) were used in these tests. Twenty-one of these cements were of the normal, non-air-entraining type then in general use and were classified as follows: eight were Type I; five, Type II; three, Type III; four, Type IV; and one, Type V. In addition, six air-entraining cements were made by intergrinding flake Vinsol resin with the clinkers used in making the corresponding non-air-entraining cements. Four of these were Type I; one, Type II; and one, Type III. In the tables and discussion which follow, the various cements are identified by number, the first digit in each case indicating the type and the second digit the number of the cement within the type. The six air-entraining cements are identified by the letter "T" immediately following the cement identification number.

Test program

With some exceptions, at least 24 of the 27 cements were used in each of a number of full-size and near full-size concrete structures located in various parts of the country. These included three concrete pavements, totaling approximately 6 miles of standard two-lane construction, one located in southwestern New York in an area subject to severe natural weathering, one in central Missouri where the exposure is moderately severe, and one in western South Carolina where mild weather conditions prevail. There are also four installations of reinforced concrete piles, three in salt water, and one in fresh. The test cements were also used in the parapet wall of an earth-filled dam; in the walkway slabs of a multiple arch dam; in test specimens subject to action of alkali soils; and in two installations of near job-size specimens—one near Chicago, where exposure conditions are severe and the other near Macon, Ga., under mild exposure conditions.

To minimize, insofar as practical, the effect of accidental variables, each cement was used a number of times in each structure. For example, on the New York test road, each cement was used in four test sections each 12 ft wide and 150 ft long. The cements were used in rotation, the sequence being such as to avoid abrupt changes in finishing characteristics. Not all of the cements were used in all of the structures. However, the entire 27 were used

approximately 80 times, with 22 cements being used 95 times. This, together with the fact that approximately 50,000 bbl of cement were required for the entire project, will give some idea of the magnitude of the operation.

During the 15 years that have elapsed since these structures were built, a number of reports showing the progress of deterioration due to weathering have been issued. Most of these are readily available in the *Proceedings* of the American Concrete Institute. Several have also been issued as bulletins by the Portland Cement Association. A list of published reports covering the project is given in the bibliography.

THE TEST ROADS

Three reports covering the test roads have already been published. The first of these appeared as a portion of a paper entitled "Why Type II Cement?" prepared by the writer in 1949.[8] This was followed by a more detailed report on the New York test road after 7½ years of service.[9] Finally, there appeared in 1953 the 10-year report on the Long-Time Study.[10] In these reports the discussion, insofar as pavement performance was concerned, was confined entirely to the New York road. The only reference to the Missouri and South Carolina roads was to point out that at the end of 10 years exposure no differences in behavior attributable to cements had developed on either of these projects.

In the present report it is proposed to discuss the condition of all three test roads as revealed by inspections made by the writer and others* during the summer of 1957. After 15 years exposure in actual service it should be possible to draw some rather definite conclusions regarding the comparative weathering characteristics of these cements. However, other factors in addition to the cement affect pavement durability. These include: (1) the physical properties and grading of the aggregates used in the concrete; (2) the proportions, including cement, water, and air content; (3) the consistency; and (4) the methods of construction, particularly methods of placing and finishing. Any comparisons of cement performance on the different roads must necessarily give consideration to these other factors.

AGGREGATES

Local aggregates were used on each project. These differed considerably in mineral composition as well as in grading, thus further complicating the comparisons of cement performance.

A natural sand meeting state specifications 47L and two sizes of crushed gravel were used in New York. The fine aggregate was a moderately coarse sand containing about equal parts of quartz and feldspar and with about 60 percent passing the No. 14 sieve and about 15 percent passing the No. 48

*The following engineers participated in the 1957 inspections:
Missouri: D. G. Gothan, Missouri Highway Department; H. Allen and G. V. Joines, Bureau of Public Roads.
South Carolina: Messrs. Herriot, Armstrong, and Beckham, South Carolina Highway Department; H. Allen and K. F. Shippey, Bureau of Public Roads; F. R. McMillan, consulting engineer; and C. C. Oleson, Portland Cement Association.
New York: W. H. Ketchum, New York Department of Public Works; Messrs. H. Allen, Groves, and Mabel, Bureau of Public Roads; and I. L. Tyler, Portland Cement Association.

sieve. Sodium sulfate loss averaged 7 percent at five cycles and 9 percent at ten cycles. The coarse aggregate was a crushed gravel of glacial origin composed of argillaceous sandstone with an appreciable amount of shale. The maximum size was 2 in. Sodium sulfate tests, run on separated sizes, averaged about 10 percent loss at five cycles and 27 percent loss at ten cycles.

On the South Carolina project two sizes of fine and one size of coarse aggregate were used. A mixture of the two sands in the proportions used in the work showed 93 percent passing the No. 16 sieve and 17 percent passing the No. 50 sieve. The combined fine aggregate was considerably finer than the sands used on either of the other projects. The coarse aggregate was a crushed granite ranging from 2 in. maximum size to No. 4. Sodium sulfate tests were run on separated sizes of the coarse aggregate with weight loss varying from a maximum of 13 percent at five cycles for the 1½–2-in. size to less than 1 percent for the ¾–1½-in. size.

Fine aggregate used on the Missouri project was a natural sand from Boonville, Mo., averaging about 75 percent passing the No. 16 sieve and about 10 percent passing the No. 50 sieve. The coarse aggregates were two sizes of rather soft, high-absorption, crushed cherty limestone, the combined grading running from 1½ in. maximum to No. 4. The coarse aggregate contained an appreciable amount of dust of fracture. Absorption averaged 2.6 percent.

COMPOSITION OF THE CONCRETE

Standard paving mixtures conforming to the requirements of the states involved were used for the non-air-entrained concrete on all of the projects. For the air-entrained concrete, the usual practice of reducing the sand content by an amount necessary to maintain approximately the same cement content as in the non-air-entrained mixes was followed. In New York, actual cement content, as determined from field measurements, varied from a minimum of 6.0 sacks per cu yd to a maximum of 6.7 sacks per cu yd, with an average of about 6.5 sacks per cu yd.* In Missouri, the cement content ranged from 5.4 to 5.9 sacks per cu yd, with an average of 5.8. The range in South Carolina was from 5.5 to 6.0, with an average of 5.9. However, differences in cement content between individual cements on any one project did not, as a rule, exceed ± 0.2 sack. Differences of this order are believed to be without significant effect on comparisons of cement performance.

Except for one group of sections where the amount of mixing water was increased to give a slump of about 5 in., the New York road program called for a 3-in. slump. Actually, due to poor water control, slump test results varied considerably, not only from section to section but also within each section. A slightly drier mix averaging 2½-in. slump with better water control was used in Missouri and South Carolina. Air contents of the non-air-entrained concrete, as calculated from the field measurements, generally averaged 1.0 percent or less. An exception to this was Cement 41. Air con-

*Variations in cement content on the New York project are discussed in considerable detail in Reference 9.

tent of concrete containing this cement averaged 2.1 percent in New York, 2.0 percent in Missouri, and 1.0 percent in South Carolina. This variation was probably due to the intermittent use of rosin as a grinding aid in the manufacture of this cement.

Air contents of the air-entrained concrete varied from 2.5 to 6.2 percent in New York, from 2.7 to 3.7 percent in Missouri, and from 3.4 to 3.6 percent in South Carolina. With the exception of concrete containing Cement 16T, the air contents of all of the air-entrained concretes were low in terms of present-day practice; the average of all values was about 3 percent.

METHODS OF CONSTRUCTION

Construction practices followed those of the states involved. Types and efficiency of the power equipment used for placing and finishing varied considerably as between jobs. An old, underpowered, double-screed finisher was used in New York. In general, this equipment was not powerful enough to properly compact concrete of less than 4-in. slump. Furthermore, the operation of the equipment was inconsistent, the number of passes varying from one to as many as seven in some cases. This variability of operation imposed an added burden on the longitudinal float that followed. In addition, the comparatively large number of low, honeycombed, or torn areas left by the transverse screeds necessitated much more than the normal amount of "carry-back," thus further increasing the burden on the finishing machines. The final finish was obtained by means of a 12 in. wide plywood belt followed by a burlap drag, and, finally, brooming with 20-in. stiff-fiber brooms. The record indicates that the amount of surface manipulation that followed the operation of power equipment was less than on the average construction operation. Cotton mats, placed as soon as the surface was hard enough to withstand damage, kept wet 3 days, and then removed, were used for curing.

In South Carolina the only power equipment used to handle the concrete, after it had been deposited on the subgrade, was an old-type two-screed finisher which served also as a spreader. Number of passes varied from two to seven, with an average of about three. On this job a "Flexplane" machine was used to install a longitudinal premolded joint after which the surface was floated with hand-operated longitudinal float. The operation of this float varied widely, depending upon the degree of hardening which had taken place before the float was brought into operation. Sometimes it was used simply as a float, at other times as a combination tamper and float. Final finishing was by longitudinal steel straightedges followed by belting with a 10-in. canvas belt and dragging with an 18-in. burlap drag. Curing was by the use of wet burlap for 10 hr followed by waterproof paper for 3 days.

Concrete on the Missouri road was distributed on the subgrade with a power-screw spreader and finished with a two-screed finishing machine with tamper operating between the screeds. Both screeds and the tamper operated during the first pass, the screeds only during the second pass. A power-

operated longitudinal float followed, usually operating close to the finisher. This in turn was followed by straightedging with 10-ft steel straightedges and an 8-in. canvas belt. The final texture was obtained by brooming. Curing was by means of wet cotton mats applied for 3 days.

CLIMATIC CONDITIONS

As previously noted, the three locations were chosen to provide natural weathering conditions varying from the quite severe climate of southwestern New York, through the moderately severe climate of central Missouri, to the comparatively mild climate of western South Carolina. The principal difference in the conditions that prevail in New York as compared to Missouri and South Carolina lies in the average annual snowfall which is as high as 95 in. in the vicinity of the New York road, compared to an average of 20 in. in Missouri and less than 1 in. in South Carolina. Total precipitation, on the other hand, averages about 50 in. per year in South Carolina as compared to 40 in. per year in Missouri and New York. Temperatures during the winter months are not greatly different in New York and Missouri, averaging only about 4 deg lower at the former location. From 1941 to 1956, the average number of days per month during which the temperature fell below 32 F for the five winter months, November through March, was 21 in Missouri as compared to 24 in New York, not a great difference. In South Carolina the average temperatures were about 10 F higher.

Due to ice conditions and to the heavy snowfall, ice control chemicals have been used quite extensively on the New York road, compared to a sparing use in Missouri and no use at all in South Carolina. It is believed that these chemicals (mostly sodium and calcium chloride) have probably contributed more to the surface scaling which has developed on certain non-air-entraining cement sections of the New York road than any other single factor. The fact that practically no scaling has developed on the Missouri road, where ice control chemicals were used only sparingly, confirms this observation. However, none of the air-entrained concrete sections on the New York Test Road have scaled, even under the heavy and repeated applications of chemically treated abrasives to which they have been subjected. This will be discussed more fully in a following section.

TRAFFIC CONDITIONS

The average total daily traffic on the three test roads, based on 1957 surveys conducted by the Bureau of Public Roads, together with the number of vehicles having gross loads of over 9 tons and the approximate percentages of gross loads over 9 tons are given in Table 1.

The average total daily traffic on the South Carolina road was over five times as great as reported for Missouri and almost four times as great as indicated for New York. However, the percentage of gross loads over 9 tons was about the same in all cases.

TABLE 1—1957 TRAFFIC ON THE TEST ROADS

Project	Total average vehicles per day of 24 hr	Gross loads over 9 tons	Approximate percent of gross loads over 9 tons
Missouri	1848	80	4.3
South Carolina	10258	301	3.0
New York	2743	90	3.3

PRESENT CONDITION OF THE TEST ROADS

New York

The New York test road constitutes Project 1 of the Long-Time Study. It is 2.6 miles long, located in southwestern New York on State Route 17, between Wellsville and Bolivar. It was constructed during late summer and fall of 1942 and has therefore gone through 15 winters. The test road is divided into three parts, designated as Project 1, 1A and 1B. Project 1 contains 24 of the test cements (all 21 of the non-air-entraining and three of the air-entraining cements) and was designed to compare them on the basis of their resistance to frost action uncomplicated by the action of chloride salts. Unfortunately, it was found necessary as early as 1945 to apply ice control chemicals to certain portions of Project 1, including the sections lying on a slight grade immediately west of the village of Allentown, as well as to certain sections east of the town. It is rather difficult to make comparisons based on the effect of natural weathering alone since precise locations in which the ice control chemicals were employed cannot be clearly defined.

Project 1A was set up to study the effect of using a somewhat wetter mix (5-in. slump) as compared to the 3-in. slump used in Project 1. Three of the non-air-entraining cements used in Project 1 together with their air-entraining counterparts (a total of six cements) were used in this project.

Project 1B contains six of the non-air-entraining cements together with their air-entraining counterparts, 12 cements in all. These sections were constructed on a fairly steep grade at the east end of the test road where it was anticipated that ice control chemicals would be used regularly.

The New York test road has been inspected approximately annually since its construction. The writer has participated in a number of these inspections, including the most recent made in June, 1957. A complete record of the behavior of the test sections including structural cracking, which has been extensive, has been obtained. However, in accordance with the primary objectives of the Long-Time Study, principal emphasis has been directed toward obtaining evidence indicative of lack of durability or weather resistance. Careful notes were taken during the earlier inspections regarding the condition of the joints, including the formation of D-cracks,* spalling, raveling, and scaling. The formation of D-cracks along construction joints and structural cracks was formerly considered quite significant as indicating a ten-

*See definition in footnote, p. 1018.

dency toward premature weathering or lack of durability of the concrete slab as a whole. However, continued observation of this and many other concrete roads has indicated that progressive deterioration following D-cracking is usually confined to the vicinity of the joint itself and does not, as a rule, progress to the interior of the slab. On the New York test road, the D-cracking noted in the earlier reports has probably progressed by now to a condition where raveling or scaling along the joint has occurred. This is the usual result of D-cracking. However, the practice of overfilling the joints with asphalt during routine maintenance operations usually leaves an excess of asphalt in the surface which has prevented any detailed examination of the concrete in this case. For this reason no attempt to rate the joints, as in previous inspections, was made in 1957. Attention of the observers was confined to recording to the nearest 5 percent the appreciable amount of surface scale that has developed on most of the non-air-entrained sections.

Table 2 shows in round figures the amount of surface scale (expressed as a percentage of the total area of the slab) that has occurred on Project 1 as of: March, 1945; August, 1951; June, 1955; and June, 1957. Although, as previously stated, no sections on this portion of the test road were supposed to have been salted, the record shows that beginning in 1945 several areas, including the grade west of Allentown, have had intermittent applications of chemically treated abrasives. Since it is impossible to identify with certainty the exact boundaries of the salt treated sections and also because the salt treated abrasives may have been tracked some distance along the road from the point of application, it is difficult to separate the salted from the unsalted portions of the pavement. However, if we assume that salts were used on the grade west of Allentown and also in the vicinity of certain intersecting roads, it is possible to spot some of the locations which were probably salted. Test sections corresponding to these locations are shown by an asterisk in Table 2. Reference to the table shows at once that most of the sections in Rounds 2 and 4 were subject to salt action and that, as a rule, the percentage of scale on these sections is considerably higher than on the sections placed during Rounds 1 and 3.

There are, however, two exceptions to this general rule. In the case of the three air-entraining cements (12T, 16T, and 21T) we find no evidence of scaling in any of the rounds in spite of the fact that sections containing these cements in Rounds 2 and 4 were subject to salting in the same manner as the non-air-entraining cements. This confirms a fact now well recognized—that air-entrained concrete is highly resistant to surface scaling resulting from alternate freezing and thawing, either with or without the accompanying action of chemically treated abrasives. The other exception is that the non-air-entraining Type IV and V cements show considerably less scale than the other non-air-entraining types. Omitting Section 51-2, average surface scale for Type IV and V cements is about 2 percent, compared to about 30 percent for the average of Type I, II, and III cements. However, note that Cement 41 is in reality a mildly air-entraining cement (2 percent air) and that

fact may explain its excellent behavior. In the case of the other three Type IV cements, and the single Type V cement, there is no evidence in the record to indicate that entrained air exceeded 1 percent in any case.

Another point of interest in Table 2 is that, in general, the Type II cements as a group show no greater resistance to scaling than the Type I cements. This statement is made with full realization of the uncertainties involved in mak-

TABLE 2—PROGRESS OF SCALING, PROJECT 1, NEW YORK TEST ROAD

Section No.	Surface scale, percent of total area of slab				Section No.	Surface scale, percent of total area of slab			
	1945	1951	1955	1957		1945	1951	1955	1957
11 -1	0	T	5	5	22 -1	0	0	T	5
2*	5	25	55	90	2	0	T	5	15
3*	0	T	20	20	3	0	0	1	5
4*	0	T	15	15	4*	0	T	10	60
12 -1	0	T	10	5	23- 1	0	1	30	50
2*	0	5	20	20	2*	0	1	25	50
3	0	T	5	10	3	0	1	35	C
4*	0	0	1	T	4*	0	T	15	30
12T-1*	0	0	0	0	24 -1	0	1	10	35
2*	0	0	0	0	2	0	1	3	5
3	0	0	0	0	3	0	2	3	10
4	0	0	0	0	4*	T	T	20	25
13 -1	0	T	10	45	25- 1	0	T	1	10
2*	15	45	90	C	2	0	T	5	20
3	0	T	5	10	3	0	0	5	T
4*	0	T	20	50	4*	0	T	1	15
5	0	—	90	90					
14 -1	0	0	0	0	31- 1	0	T	30	60
2	0	T	5	5	2*	T	1	30	55
3	0	1	5	10	3	0	0	T	5
4*	0	1	5	10	4*	0	0	T	5
15 -1	0	T	5	5	33- 1	0	0	1	5
2*	1	15	50	90	2*	0	T	5	15
3	0	T	10	15	3	0	T	5	15
4*	0	T	40	C	4	0	0	0	T
16 -1	0	1	10	15	34 -1	0	T	20	30
2*	0	2	15	30	2*	0	15	70	C
3	0	5	25	30	3	0	T	30	35
4*	0	T	10	20	4*	0	T	25	40
5*	0	—	10	20					
16T-1	0	0	0	0	41- 1	0	0	0	0
2*	0	0	0	0	2*	0	0	0	T
3	0	0	0	0	3	0	0	0	0
4*	0	0	0	0	4	0	T	0	0
17 -1	0	T	10	20	42 -1	0	0	0	T
2*	0	5	60	C	2	0	T	1	T
3	0	T	15	20	3	0	8	0	0
4	0	T	T	5	4*	0	T	T	10
18 -1	0	T	1	T	43 -1	0	0	0	T
2*	0	5	25	50	2*	0	0	T	5
3	0	0	T	T	3	0	0	0	0
4*	0	T	10	55	4*	0	T	5	5
21- 1	0	T	1	35	43A-1	0	T	T	T
2*	0	T	10	15	2*	0	0	2	10
3	0	0	5	T	3	0	0	0	0
4*	0	0	5	35	4	0	T	0	T
21T-1	0	0	0	0	51 -1	0	0	0	T
2	0	0	0	0	2*	0	0	0	80†
3	0	0	0	0	3	0	T	1	5
4	0	0	0	0	4*	0	0	T	T

NOTE: T—trace
C—covered with bituminous materials
*Abrasives containing chemicals were probably used on this section.
†Appearance of section would indicate the condition is the result of light surface wear rather than salt scaling.

TABLE 3—PROGRESS OF SCALING, PROJECT 1A, NEW YORK TEST ROAD

Section No.	Surface scale, percent of total area of slab				Section No.	Surface scale, percent of total area of slab			
	1945	1951	1955	1957		1945	1951	1955	1957
A12 -1	0	1	20	70	A16T-1*	0	0	0	0
2*	0	1	10	30	2	0	0	0	0
3	0	2	25	35	3	0	0	0	0
4*	0	T	20	20	4*	0	0	0	0
A12T-1	0	0	0	0	A21 -1	T	1	70	C
2*	0	0	0	0	2*	0	T	50	C
3	0	0	0	0	3	0	2	50	90
4*	0	0	0	0	4*	0	1	70	C
A16 -1	0	1	20	60	A21T-1	0	0	0	0
2*	0	1	25	70	2*	0	0	0	0
3	0	10	60	80	3	0	0	0	0
4*	0	1	35	55	4*	0	0	0	0

NOTE: T—trace
C—covered with bituminous materials
*Abrasives containing chemicals were probably used on this section.

ing comparisons of the data shown. However, there is no evidence to indicate that the Type II cements were subjected either to more frequent or to heavier applications of salt treated abrasives (particularly in Round 1) than the Type I cements. Consequently, it is believed that the comparison can be considered reasonably valid.

In general, the same comments may be made with regard to the sections comprising Project 1A (5-in. slump concrete) as were made in the case of Project 1. Three of the non-air-entraining cements and their air-entraining counterparts were used in this project. The percentages of scale that had developed in Project 1A at the same four periods as shown in Table 2, are given in Table 3. It will be observed that the 12 sections containing the non-air-entraining cements have scaled badly in all four rounds whereas the corresponding 12 sections containing the air-entraining counterparts of these same cements are completely free from scale. The amount of scaling in the non-air-entraining cements in all cases is considerably higher than is shown for those same cements in Project 1 (see Table 2). This is probably due to the higher slump (and consequently higher water content) of the concrete in Project 1A. However, the outstanding fact again is the excellent performance of the air-entraining cements, none of which have shown any signs of scaling up to this time even in concrete of relatively high water content.

As previously noted, Project 1B was constructed on a considerable grade at the east end of the test road. The purpose here was to provide a comparison between the cements under conditions involving the use of chemically treated abrasives for ice control. Six of the non-air-entraining cements and the six corresponding air-entraining cements (12 cements in all) were used on this portion of the test road. All have been subjected to repeated and heavy applications of abrasives containing ice-control chemicals. The amount of scaling that has developed is shown in Table 4. Here again the principal point to be noted is the complete absence of scaling in all of the sections containing the six air-entraining cements as compared to serious scaling on

TABLE 4—PROGRESS OF SCALING, PROJECT 1B, NEW YORK TEST ROAD

Section No.	Surface scale, percent of total area of slab				Section No.	Surface scale, percent of total area of slab			
	1945	1951	1955	1957		1945	1951	1955	1957
B11 -1	10	45	85	85	B18 -1	5	25	45	60
2	1	15	30	45	2	5	1	10	20
3	T	5	10	20	3	T	10	20	30
4	T	5	10	30	4	T	5	10	10
B11T-1	0	0	0	0	B18T-1	0	0	0	0
2	0	0	0	0	2	0	0	0	0
3	0	0	0	0	3	0	0	0	0
4	0	0	0	0	4	0	0	0	0
B12 -1	15	30	75	80	B21 -1	5	30	80	C
2	2	20	50	70	2	1	5	80	C
3	T	T	5	10	3	0	5	45	C
4	T	50	C	C	4	T	30	C	C
B12T-1	0	0	0	0	B21T-1	0	0	0	0
2	0	0	0	0	2	0	0	0	0
3	0	0	0	0	3	0	0	0	0
4	0	0	0	0	4	0	0	0	0
B16 -1	15	50	90	85	B33 -1	5	45	80	C
2	5	15	40	60	2	T	20	65	60
3	T	25	40	60	3	0	0	2	5
4	T	15	40	60	4	T	1	5	10
B16T-1	0	0	0	0	B33T-1	0	0	0	0
2	0	0	0	0	2	0	0	0	0
3	0	0	0	0	3	0	0	0	0
4	0	0	0	0	4	0	0	0	0

NOTE: Chemically treated abrasives have been applied to all of the Sections of Project 1B.
T—trace
C—covered with bituminous materials

almost all of the sections containing the corresponding six non-air-entraining cements. The contrast is so great as to completely overshadow the relatively minor differences in scaling resistance indicated by the various non-air-entraining cements. This project furnishes further proof of the outstanding performance of air-entrained concrete under conditions where chemically treated sand or cinders are used extensively for ice control.

South Carolina test road

This road is Project No. 2 of the Long-Time Study. It is located in the western part of the state just southeast of Greenville. All 21 of the non-air-entraining cements and the air-entraining counterparts of three of these cements were used four times in sections 22 ft wide and 90 ft long. This made 96 test sections, each divided by contraction joints into three subsections 30 ft long, thus providing two interior joints in each test section. The entire test pavement, approximately 1.6 miles long, was constructed during the late fall of 1941, and is now 16 years old. Traffic and weather conditions which prevail on this road have already been described.

Due to the comparatively mild climate of western South Carolina, this test pavement has not been subjected to anything like the severe weather conditions that prevail in western New York where many alternations of freezing and thawing and an average annual snowfall of 90 in. may be expected. Snowfall in South Carolina is rare and the winter temperatures average several degrees higher. For these reasons certain types of defects

observed in New York, such as surface scaling and D-cracking along joints, are almost entirely absent in South Carolina. However, structural failures—that is, longitudinal and transverse cracking, corner breaks, etc.—due to traffic loads are quite frequent. Such failures are not related in any way to the weathering characteristics of the cements.

Branch and pattern cracking in some of the sections has been noted from time to time. These are the fine cracks which are often observed on flat concrete surfaces. They are most noticeable when the concrete is moist. This type of cracking does not appear to be associated with any particular cement or cement type. It is minor in extent, except in the case of Cement No. 24. In this case, Rounds 1 and 2 have been reported as affected to a marked degree whereas Rounds 3 and 4 are entirely clear. Also, there appears to have been a slight increase with time in the amount and degree of pattern cracking on Sections 24-1 and 24-2 but the concrete otherwise is still in good condition, no special maintenance measures having yet been required.

According to the 1957 inspection, tracking of bituminous material from recently placed bituminous shoulders onto the pavement has, in some cases, completely obscured the indications of surface crazing that had been noted in the earlier reports. In other cases, the concrete surface, especially the outer 3 ft, presented a somewhat mottled appearance consisting of many high and low spots. Tracking by vehicles has discolored the high spots, leaving the low spots clear, thereby accentuating the appearance of roughness. There is a distinct question in this case whether these were actually scaled areas or whether the roughened appearance was caused by wear or abrasion. Location of the test road in a mild climate with few cycles of freezing and thawing would indicate that the roughened areas probably result from wear. These areas were noted at various places along the road and did not appear to be associated with either individual cements or cement types, except that the Type III cements appeared to be free from this type of defect.

In general, the South Carolina test road is still in excellent condition. The minor defects noted in no way affect the serviceability of the road; it should give good service for many years.

Missouri test road

This is Project 3 of the Long-Time Study. The test road is located in the east-central section of the state, on Route 22 about 3 miles west of Mexico, Mo. All 21 of the non-air-entraining cements and the air-entraining counterparts of three of these cements were used in this test. Each cement was used five times in full-width sections, 22 x 75 ft, with a transverse plane of weakness in the center of each test section. This made a total test length of 9000 ft or 1.8 miles. The road, built during the summer of 1941, is now 16 years old. Traffic and weather conditions prevailing have already been described.

Several inspections of the Missouri test road have been made. The first

TABLE 5—MISSOURI TEST ROAD D-CRACKING AS OBSERVED IN 1957

Cement	Round 1	2	3	4	5	Cement	Round 1	2	3	4	5
11	O	O	X	X	X	31	O	O	O	O	O
12	X	O	X	O	X	33	O	O	O	O	O
12T	O	O	O	O	O	34	O	O	O	O	O
13	O	X	O	O	O						
14	O	O	X	X	NR	41	O	O	O	O	O
15	X	O	NR	O	O	42	O	O	O	X	X
16	O	O	O	O	O	43	O	O	O	O	O
16T	O	O	O	O	O	43A	O	O	O	O	X
17	O	O	O	X	NR						
18	O	O	O	O	O	51	O	O	O	O	O
21	X	O	X	X	X						
21T	O	O	O	O	O						
22	O	O	X	O	O						
23	X	O	O	X	X						
24	O	O	O	O	O						
25	O	O	X	O	X						

NOTE: O—No D-cracking or scale observed in 1957
X—D-cracking and/or scale observed in 1957
NR—No record

complete inspection was in 1942, 1 year after the road was built, the most recent in June, 1957. The pavement is still in excellent condition after 16 years service. Early inspections revealed numerous chert "pops" in the surface of the pavement. These pops do not appear to affect the structural strength or general durability of the concrete. Chert pops are almost completely absent on all sections containing the Type III cements. Otherwise they are fairly well distributed among the cements, regardless of type.

Aside from chert pops, observable defects in the Missouri test road were confined to a small amount of spalling along the joints. Some faint D-cracking was also observed at the joints; its distribution by cements and rounds is shown in Table 5. The three air-entraining cements as well as all three of the Type III and the one Type V non-air-entraining cements were completely free from D-cracking at the time of this inspection. This is also true of Type IV cements, 41 and 43. On the other hand, only three of the Type I and Type II non-air-entraining cements are completely free from D-cracking in all rounds. These are Cements 16, 18, and 24.

Defects thus far observed on the Missouri test road are not serious enough to affect the serviceability of the pavement, which is in excellent condition after 16 years use. There are practically no structural defects and the chert pops, although numerous and unsightly, have had no other adverse effects. Joint spalling is not serious enough to warrant concern, provided the joints are given proper maintenance.

SUMMARY

Project 1, the New York test road

The progress report[9] issued in 1951 states that "all six of the air-entraining cements are definitely superior to the non-air-entraining cements in resistance to surface scaling." This statement still held true in 1957. In fact, the

contrast is even more striking now due to the progressive scaling that has occurred on most of the non-air-entrained sections as compared to no scaling at all on any of the sections containing the air-entraining cements.

Except for the Types IV and V cements, there is still no evidence to indicate that any one non-air-entraining cement or any one type of non-air-entraining cement is consistently more resistant to scaling than another. In general, there is a greater difference in the amount of scale from round to round for a given cement than there is between the averages for the various cements. The Type II cements show up no better in this regard than Type I cements, a fact that may surprise many of those who have held quite firm opinions to the contrary. On the other hand, the data do show that the Type IV and V cements are much more scale resistant than the other non-air-entraining types. However, it would seem unwise to draw any general conclusion regarding the superior scale resistance of the Type IV and Type V cements based on the evidence of the New York test road alone.

The generally inconsistent behavior of many of the non-air-entraining cements as regards frost resistance suggests the probability that physical characteristics of the concrete are more important in determining its ultimate durability than is the chemical composition of the cement. This view is supported by other research studies which indicate that the physical structure of the cement paste, particularly the number of air voids per unit of volume of the paste, is a controlling factor in the matter of durability. The results of these studies should throw considerable light on the problem of reconciling certain anomalies with which we are confronted in our study of cement performance on these projects. In the meantime, we should not overlook the fact that the influence of such factors as chemical composition and fineness is, in reality, of minor significance as compared to the great improvement which results from the use of air-entrained concrete under conditions where chemicals will be used for ice removal. This is the really important lesson taught by the New York test road.

Projects 2 and 3, the South Carolina and Missouri test roads

The South Carolina and Missouri test roads are both in excellent condition after 16 years service. Such surface defects as have developed are minor and, so far, have necessitated no special maintenance measures. Sections containing both the air-entraining and the non-air-entraining cements are equally good. There has been some pattern cracking and some intermittent light scale or surface wear on the South Carolina road and a considerable number of chert pops on the Missouri road. There is some spalling as well as some D-cracking on this road also. However, none of these defects can be associated in any way with a particular cement or a particular type of cement, with the following exceptions. On the South Carolina road one of the Type II cements, No. 24, developed more than the average amount of pattern cracking in two of the four rounds containing this cement. Also, all of the sections containing the Type III cements were entirely free from the light

scale or wear observed on the South Carolina road as well as entirely free from the chert pops and D-cracking observed in Missouri.

The good record of the Type III cements in South Carolina and Missouri and the correspondingly good record of the Type IV and Type V cements in New York again emphasizes the difficulty of drawing any general conclusions regarding the superior durability of any one type of cement as compared to another. Aside from the exceptions noted, all of the cements have performed equally well on both projects. On the basis of this performance, it would certainly seem safe to assume that, under similar conditions of exposure, variations in the chemical composition and fineness of cements within the limits represented by this study, are without practical significance, insofar as frost resistance is concerned.

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Title No. 54-60

Static and Fatigue Strength in Shear of Beams with Tensile Reinforcement*

By TIEN S. CHANG† and CLYDE E. KESLER‡

SYNOPSIS

Dimensional analysis is employed in deriving expressions for the initial diagonal cracking load and the ultimate strength in shear of simply supported concrete beams with tensile reinforcement only. These expressions, which include size effect, are converted into nomographs for ease of application. This study of static strength includes results from tests of 105 beams, 42 of which were tested by the authors.

Fatigue tests were made on 39 reinforced concrete beams with tension reinforcement only. These beams were simply supported on a span of 60 in. and loaded at the third points. Statistical studies of the fatigue behavior with regard to initial diagonal cracking and final failure are included.

INTRODUCTION

Static loading

Since the derivation of the familiar expression $v = V/bjd$§ by Mörsch[1] in 1903, investigations which have been made on reinforced concrete beams failing in shear were generally interpreted by expressing the ultimate nominal unit shearing stress, v_u or V_u/bjd, as a function of certain variables. In 1909, Talbot[2] indicated that the ultimate nominal unit shearing stress depends on many variables, such as span-depth ratio, concrete strength, percentage of tensile reinforcement, and amount of web reinforcement. Consideration of these variables, with the exception of concrete strength and web reinforcement, have generally been neglected by most subsequent investigators when determining empirical expressions of the ultimate nominal unit shearing stress. Some earlier interpretations even neglected the effect due to the variation of concrete strength. Tests have been made to determine the limiting nominal unit shearing stress, using the expression:

$$\left.\begin{aligned} v_u &= V_u/bjd = Krf'_{sp} + cf_c' \\ \text{or } v_u &= cf_c' \text{ for beams without web reinforcement} \end{aligned}\right\} \quad \text{(a)}$$

and results thus obtained were varied and inconsistent.

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†Member American Concrete Institute, Associate Professor of Applied Mechanics, Virginia Polytechnic Institute, Blacksburg, Va. (Formerly Assistant Professor of Theoretical and Applied Mechanics, University of Illinois).

‡Member American Concrete Institute, Professor of Theoretical and Applied Mechanics, University of Illinois, Urbana, Ill.

§Notations are summarized in Appendix 1.

In 1945 Moretto[3] attempted to include other variables which might have an influence on the ultimate nominal unit shearing stress of a reinforced concrete beam failing in shear, and from his data developed the following empirical formula:

$$\left.\begin{aligned} v_u &= V_u/bjd = Krf'_{yp} + 0.10f_c' + 5000p \\ \text{or } v_u &= 0.10f_c' + 5000p \text{ (without web reinforcement)} \end{aligned}\right\} \quad \text{(b)}$$

A new variable, steel percentage, was included and the expression fitted his experimental data fairly well.

In 1951 Clark[4] fitted the following expression to his experimental data:

$$\left.\begin{aligned} v_u &= V_u/bjd = 2500\sqrt{r} + 0.12f_c'(d/a') + 7000p \\ \text{or } v_u &= 0.12f_c'(d/a') + 7000p \text{ (without web reinforcement)} \end{aligned}\right\} \quad \text{(c)}$$

Another variable, the depth-span ratio, has been added to this formula. Although both Moretto's and Clark's formulas fit their own experimental data fairly well, the formulas did not give comparable results.

In April, 1953, Laupa, Siess, and Newmark[5] considered the actual behavior of a reinforced concrete beam without web reinforcement under combined flexure and shear, and found that generally the beam cracked diagonally, and then failed by the destruction of concrete in the compression zone.* They therefore assumed that the final failure of a reinforced concrete beam without web reinforcement due to combined flexure and shear is essentially the same as that due to flexure, the only difference being that the diagonal crack concentrates the angle changes that would normally be distributed along a considerable length of the beam to a relatively short section at the end of the diagonal crack. Therefore, the maximum moment (shear-moment) the beam can carry with a diagonal crack will be much smaller than in the case for pure flexure. Using this shear-compression theory, an empirical equation expressing the shear-moment for beams without web reinforcement was developed:

$$M_s = kbd^2f_c'(0.57 - 4.5f_c'/10^5) \quad \text{(d)}$$

In June, 1953, Moody, and in 1954 and 1955 Moody, Viest, Elstner, and Hognestad[6] presented a semiempirical formula, based on the same shear-compression failure theory, to express the shear-moment that a concrete beam with tensile reinforcement can carry. They assumed that the moment could be expressed by the same type of equation as for pure flexure, namely:

$$M_s = A_sf_sd\left(1 - k_2\frac{pf_s}{k_1k_3f_c'}\right) \quad \text{(e)}$$

*Actually the part of beam where final failure occurs is subjected to combined compression and shear; but if the span-depth ratio is large enough, the total force is approximately the same as the compressive force.

Assuming $k_1k_3 = 1.121 - 0.0485\, f_c'/1000$, $k_2 = 0.42$, and that the steel stress could be determined from a similar expression as that for over-reinforced concrete beam in pure flexure, the following empirical expression for steel stress at failure in the beam weak in shear was developed:

$$f_s = 15{,}080\left(-1 + \sqrt{1 + \frac{1450}{E_s p/k_1k_3 f_c'}}\right) \quad \text{.....(f)}$$

Both Laupa's and Moody's formulas give good predictions of the ultimate shear-moment for beams with relatively large cross sections ($d \geqq 8$ in.). These formulas did not, however, apply to beams with relatively small cross sections ($d \leqq 8$ in.).

Since development of the shear-compression failure theory, investigations have been made and are now under way to study the effects of size of cross section, web reinforcement, compressive reinforcement, continuous supports, uniform loading, and repeated loading. Some of these variables have already been treated in Laupa's and Moody's papers.

The shear-moment will control the ultimate load carrying capacity of a beam weak in shear only if the beam will crack diagonally before the shear-moment of the beam is reached. A beam that cracks diagonally at a load higher than that predicted by the shear-moment theory will therefore take a higher load than that which produces the shear-moment. However, as soon as a diagonal tension crack is fully developed, the beam will fail since the moment on the beam at the critical section is already higher than that of the shear-moment. Moody developed a formula that will predict the load for the initiation of diagonal tension cracking of a beam as determined from load-deflection curves. The predicted load is generally lower than that at which the diagonal tension cracks can be determined with the unaided eye. This formula involves both the concrete strength and the span-depth ratio, a/d or M/Vd, and is as follows:

$$\left.\begin{aligned} &\text{For } 1000 \text{ psi} < f_c' \leqq 5000 \text{ psi}: \\ &\qquad v_c = V_c/(\tfrac{7}{8}\, bd) = 0.12\,(1 - 0.1M/Vd)\, f_c'\,(1 - f_c'/10{,}000) \\ &\text{For } f_c' > 5000 \text{ psi}: \\ &\qquad v_c = (v_c)_{f_c' = 5000} \end{aligned}\right\} \quad \text{.....(g)}$$

Morrow[7] in 1954 considered the cracking of a beam weak in shear as a problem of combined stresses, and developed a formula which predicts the initial diagonal cracking load of knee frames without web reinforcement and of the beams tested by Moody. This formula is as follows:

$$v_c/f_r = \frac{13.4}{\frac{a_c}{npd} + \sqrt{\left(\frac{a_c}{npd}\right)^2 + 600}} \quad \text{(h)}$$

$$\text{with } f_r = 0.275\left(1 - \frac{f_c'}{8250}\right) \text{ for } f_c' \leqq 3000 \text{ psi}$$

$$f_r = 300 + 0.075 f_c' \text{ for } f_c' \geqq 3000 \text{ psi}$$

$$a_c = a'/2 \text{ for } a/d \geqq 2$$

$$a_c = d \text{ for } a/d < 2$$

$$\text{and } E_c = 1{,}800{,}000 + 460 f_c'; \quad E_s = 28{,}000{,}000 \text{ psi}$$

None of the preceding formulas were found to be applicable to the beams with relatively small cross sections used in this investigation. Nevertheless a satisfactory means of determining the static strength of beams is necessary before the results of repeated load tests made by the authors can be evaluated.

Fatigue loading

The fatigue strength of reinforced concrete beams is a complicated subject about which little is known. Variables, such as concrete strength, percentage of tensile, compressive, and web reinforcement, maximum value of repeated loading, range and rate of loading, aggregate size and shape, temperature, humidity, and others, will have significant effects on the final test results. If a beam subjected to repeated loading is weaker in shear than flexure, it may fail in any one of the following five ways: (1) destruction of the compressive zone, (2) splitting action along the reinforcement, (3) diagonal cracking, (4) bond failure, or (5) fatigue of the reinforcement.

Generally, a beam weak in shear and subjected to repeated load will first crack diagonally and the cracking will then proceed toward the point of maximum moment as additional loads of the same magnitude are applied. The beam will fail when the crack extends so far that the compression zone becomes too small to resist the compressive force acting on it at the maximum value of the cyclic loading. Failure then occurs by destruction of the compression zone.* It has been observed that some splitting action along the reinforcing bars also takes place as the diagonal crack progresses, but this appears to be only of secondary importance. It has been found that bond failures are unlikely with deformed bars meeting ASTM specification A 305† for the l/d ratio tested. Another interesting phenomenon in a beam under repeated loading is the continual change in stiffness of the beam which is believed to be due to the creep behavior of concrete, the development of the initial vertical tensile cracks, and the formation of the diagonal cracks.

*Note that unlike the static case, the beam may fail by destruction of the compression zone, even when the beam has a larger predicted cracking load than the shear-moment, because the crack may, under repeated load, occur at a load lower than the load for cracking if loaded statically.

†"Minimum Requirements for the Deformations of Deformed Steel Bars for Concrete Reinforcement," A 305-53T, *1955 Book of ASTM Standards*, Part 1, pp. 1020-1022.

DETAILS OF INVESTIGATION

Scope

The purpose of this study was twofold: first, to develop simple and sufficient expressions for the static strength of beams failing in shear; and second, to determine the strength under repeated loading of reinforced concrete beams weak in shear. Emphasis was on small beams with tensile reinforcement only. The variables studied in fatigue are limited to concrete strength, percentage of reinforcement, and magnitude of the repeated load.

Outline of tests

Forty-two simply supported beams of relatively small cross sections with tensile reinforcement only were tested to failure under static loads. All beams were 4 x 6 in. in cross section, but the values of shear span, steel percentage, and concrete strength were varied as shown in Table 1. The loads at which diagonal tension cracks occurred and the maximum loads were observed. All beams with few exceptions were tested at an age of approximately 28 days, and in all cases the control cylinders were tested on the same dates as the beams.

Data on the shear strength of 64 beams of relatively large cross sections without web or compressive reinforcement were obtained from reports by Moody,[6] Richart,[8] Richart and

TABLE 1—DATA AND RESULTS OF ANALYSIS OF STATIC TESTS OF AUTHORS' BEAMS*

Beam No.	f_c', psi	p, percent	a, in.	V_c (test), lb	$\frac{V_c \text{ (test)}\dagger}{V_c \text{ (calc.)}}$	M_s (test), in.-lb.	$\frac{M_s \text{ (test)}}{M_s \text{ (calc.)}}$
IA1	4000	2.89	21	4470	0.86	104,500	1.13
IA2	4000	2.89	10	....	....	101,500	1.10
IB1	4000	1.86	13	4400	0.93	62,700	0.87
IB2	4000	1.86	13	....	....	71,900	1.00
IC1	4000	2.37	17	4410	0.99	75,200	0.90
IC2	4000	2.37	17	4000	0.90	80,100	0.96
IIA1	2560	1.86	17	3830	1.05	65,300	1.07
IIA2	2560	1.86	17	3880	1.07	66,300	1.09
IIB1	2560	2.37	21	3730	1.00	83,600	1.18
IIB2	2560	2.37	10	....	....	80,400	1.14
IIC1	2560	2.89	13	4000	0.85	82,200	1.06
IIIA1	2160	2.37	21	3830	1.13	76,000	1.13
IIIA2	2160	2.37	10	4650	1.15	74,300	1.11
IIIB1	2160	1.86	17	3450	1.04	58,600	1.02
IIIB2	2160	1.86	17	3480	1.05	59,200	1.03
IIIC1	2160	2.89	13	4130	0.95	82,600	1.13
IIIC2	2160	2.89	13	....	....	77,200	1.06
4-1a	5860	1.86	19	....	....	95,000	1.12
4-2a	5210	1.86	19	....	....	85,500	1.07
4-3a	5540	1.86	19	....	....	85,500	1.03
4-4a	5940	1.86	19	....	....	95,500	1.11
4-5a	5730	1.86	19	....	....	85,500	1.01
4-13a	4930	1.86	19	....	....	72,000	0.92
4-13b	4930	1.86	19	....	....	66,500	0.85
4-14a	5410	1.86	19	....	....	85,500	1.04
4-14b	5410	1.86	19	....	....	91,200	1.11
4-21a	5600	1.86	19	4740	0.97	.....	....
4-21b	5600	1.86	19	5540	1.13	.....	....
4-22a	4620	1.86	19	4820	1.05	.....	....
4-22b	4600	1.86	19	5270	1.15	.....	....
4-23a	4670	1.86	19	4860	1.05	.....	....
4-23b	4670	1.86	19	5060	1.10	.....	....
5-7a	4680	2.89	19	....	....	89,400	0.90
5-7b	4680	2.89	19	....	....	89,000	0.90
5-9a	4470	2.89	19	....	....	87,900	0.90
5-9b	4470	2.89	19	....	....	89,800	0.92
5-21a	4670	2.89	19	6490	1.15	.....	....
5-21b	4670	2.89	19	6180	1.10	.....	....
5-22a	4520	2.89	19	5040	0.91	.....	....
5-22b	4520	2.89	19	5830	1.04	.....	....
5-23a	4650	2.89	19	5510	0.98	.....	....
5-23b	4650	2.89	19	5250	0.93	.....	....

*For all beams: b = 4 in. and d = 5.38 in.
†In computing V_c, $a' = (a + 1)$ in. is used instead of a.

TABLE 2—DATA AND RESULTS OF ANALYSIS OF MOODY'S BEAMS: STATIC TESTS

Beam No.	f_c', psi	p, percent	b, in.	d, in.	a^*, in.	V_c (test), lb	$\frac{V_c \text{ (test)}}{V_c \text{ (calc.)}}$	M_s (test), in.-lb	$\frac{M_s \text{ (test)}}{M_s \text{ (calc.)}}$
A1	4400	2.17	7	10.30	29.5	13,000	0.93	399,000	0.81
A2	4500	2.15	7	10.50	29.5	15,000	1.01	443,000	0.89
A3	4500	2.22	7	10.55	29.5	14,000	0.98	502,000	0.95
A4	4570	2.37	7	10.63	29.5	15,000	1.00	472,000	0.84
B1	3070	1.62	7	10.50	29.5	11,450	1.00	374,000	1.00
B2	3130	1.63	7	10.55	29.5	13,500	1.17	399,000	1.04
B3	2790	1.60	7	10.63	29.5	12,000	1.09	369,000	1.05
B4	2430	1.66	7	10.69	29.5	11,900	1.14	369,000	1.05
C1	920	0.81	7	10.55	29.5	4,500	0.87	.....	....
C2	880	0.83	7	10.70	29.5	5,500	1.05	.....	....
C3	1000	0.80	7	10.75	29.5	5,700	1.02	.....	....
C4	980	0.82	7	10.80	29.5	5,650	1.01	.....	....
1	5320	1.89	6	10.56	34.0	11,500	0.95	442,000	1.00
2	2420	1.89	6	10.56	34.0	7,500	0.86	272,000	0.85
3	3740	1.89	6	10.56	34.0	11,500	1.08	400,000	1.05
4	2230	1.89	6	10.56	34.0	8,500	1.00	308,000	0.99
5	4450	1.89	6	10.56	34.0	10,500	0.92	399,000	0.97
6	2290	1.89	6	10.56	34.0	7,500	0.88	264,000	0.84
7	4480	1.89	6	10.56	34.0	10,000	0.88	391,000	0.95
8	1770	1.89	6	10.56	34.0	7,000	0.95	238,000	0.83
9	5970	1.89	6	10.56	34.0	11,500	0.93	408,000	0.87
10	3470	1.89	6	10.56	34.0	9,500	0.92	374,000	0.98
11	5530	1.89	6	10.56	34.0	11,000	0.90	460,000	1.02
12	2930	1.89	6	10.56	34.0	10,500	1.10	361,000	1.04
13	5480	1.89	6	10.56	34.0	10,000	0.83	425,000	0.95
14	3270	1.89	6	10.56	34.0	9,700	0.97	330,000	0.90
15	5420	1.89	6	10.56	34.0	11,500	0.95	391,000	0.88
16	2370	1.89	6	10.56	34.0	8,000	0.93	289,000	0.91

*In computing V_c, $a' = (a + 2)$ in. is used instead of a.

Jensen,[9] Gaston, Siess, and Newmark,[10] Clark,[4] and Laupa, Siess and Newmark,[5] and are given in Tables 2 and 3.

Thirty-nine beams having 4 x 6-in. cross sections with tensile reinforcement only were subjected to repeated loading. The results of these tests are given in Table 4. The tests were generally continued until final failure occurred. Those that did not fail at a fatigue life of about 10,000,000 cycles were removed from the testing machine and tested to failure statically. Data from this series of beams are given in Table 5.

Materials

The aggregates used in the tests made by the authors were from the Wabash River near Covington, Ind. These aggregates are part of a glacial outwash, probably from the Wisconsin glaciation. The sand had an average fineness modulus of 3.0. The coarse aggregate was well graded and had a maximum size of 1 in. The absorption of the aggregates was about 1 percent by weight of the surface dry aggregates. Both of these aggregates passed the usual ASTM specification tests. The cement used was a Type I portland cement, and was of several brands. Concrete strengths varied from 3150 to 6750 psi, with slumps varying from 1 to 6 in. Both a pan-type mixer of about 2 cu ft capacity and a free-falling nontilting drum mixer of about 6 cu ft capacity were used. The concrete was cured in a moist room at 74 F and 100 percent relative humidity for 6-7 days, and then cured in the laboratory atmosphere until tested.

The reinforcement consisted of deformed bars of intermediate grade billet steel meeting ASTM specification A 305. The average yield point was about 47,500 psi, the ultimate strength about 72,500 psi, and the modulus of elasticity about 27×10^6 psi.

Materials used in tests by other investigators varied considerably and the reader is referred to the original reports for their properties.

Testing procedures

Beams designed for the study of size effect were tested statically in a 300,000-lb testing machine under center- or third-point loading while simply supported. Each test lasted from 15 to 60 min.

TABLE 3—DATA AND RESULTS OF ANALYSIS OF CLARK'S, LAUPA'S, RICHART'S, AND GASTON'S BEAMS: STATIC TESTS

Beam No.	f'_c, psi	p, percent	b, in.	d, in.	M_s (test), in.-lb	$\frac{M_s\ (test)}{M_s\ (calc.)}$
Reference 4—Clark						
AO-1	3120	0.98	8	15.37	686,000	0.89
AO-2	3770	0.98	8	15.37	830,000	1.00
BO-1	3420	0.98	8	15.37	770,000	0.96
BO-3	3410	0.98	8	15.37	810,000	1.01
CO-1	3580	0.98	8	15.37	832,000	1.02
CO-3	3420	0.98	8	15.37	798,000	1.00
DO-1	3750	0.98	8	15.37	810,000	0.98
DO-3	3760	0.98	8	15.37	825,000	1.00
Reference 5—Laupa, Siess, and Newmark						
S-2	3900	2.08	6	10.58	458,000	1.12
S-3	4690	2.52	6	10.44	574,000	1.17
S-4	4470	3.21	6	10.37	600,000	1.12
S-5	4330	4.11	6	10.31	538,000	0.96
S-11	2140	1.90	6	10.51	365,000	1.15
S-13	3800	4.11	6	10.31	538,000	1.02
Reference 8—Richart						
291.1	1690	1.65	8	10	304,000	0.98
291.2	1690	1.65	8	10	270,000	0.87
291.3	1690	1.65	8	10	333,000	1.07
294.1	1490	1.65	8	10	300,000	0.99
294.2	1490	1.65	8	10	243,000	0.81
293.4	2350	1.65	8	10	329,000	0.93
293.5	2350	1.65	8	10	417,000	1.17
221.1	4076	2.33	8	21	2,690,000	1.16
221.2	3696	2.33	8	21	2,670,000	1.18
222.1	4522	2.33	8	21	2,980,000	1.19
222.2	4337	2.33	8	21	2,270,000	0.93
Reference 9—Richart and Jensen						
1	4760	2.80	8	21	2,290,000	0.82
2	4620	2.80	8	21	2,550,000	0.91
3	4290	2.80	8	21	2,430,000	0.91
4	3860	2.80	8	21	2,150,000	0.83
5	2230	2.80	8	21	1,690,000	0.82
6	2630	2.80	8	21	1,865,000	0.85
Reference 10—Gaston, Siess, and Newmark						
T2Ma	4320	1.38	6	10.58	332,000	0.95
T2Mb	4020	1.38	6	10.58	352,000	1.04
T2Mc	4470	1.90	6	10.58	450,000	1.09

Beams for the fatigue studies were tested in a specially designed fatigue machine which applied a sinusoidal repeated load at 440 cpm at the third points of a 5-ft simple span. The maximum value of the repeated loading was maintained by checking the loads at reasonable intervals of time and making adjustments as required. An automatic shut-off stopped the machine when the specimen failed, and the number of cycles to failure was read from a revolution counter. For a detailed description of this fatigue machine, readers are referred to a previous paper.[11]

EXPRESSION FOR THE INITIAL DIAGONAL TENSION CRACKING LOAD

The initial diagonal tension cracking loads obtained from the authors' static beam tests were compared with values given by Morrow's formula, Eq. (h); about one-third of the results were found to differ from the formula by about ± 30 percent. A new formula for determining the initial diagonal tension cracking load of simply supported concrete beams with tensile rein-

TABLE 4—RESULTS OF FATIGUE TESTS

Beam No.	f'_c, psi	p, percent	P, lb	P/P_c	Cracking life, cycles	P/P_s	Failure life, cycles
4-3*	4580	1.86	5000	0.54	519,200	0.61	2,114,500
4-6	4250	1.86	7000	0.78	400	0.89	800
4-7	4650	1.86	8000	0.87	200	0.98	200
4-8	4640	1.86	6000	0.65	425,000	0.74	557,000
4-9	3910	1.86	6000	0.69	1,500	0.79	16,800
4-11	3950	1.86	4270	0.49	292,800	0.56	1,900,200
4-12	4290	1.86	5000	0.56	5,700	0.63	1,142,100
4-14	5410	1.86	5500	0.56	8,000	0.63	19,300
4-15	2150	1.86	5500	0.87	90	0.91	1,940
4-16	4240	1.86	6750	0.76	280	0.86	440
4-17	4300	1.86	7130	0.79	80	0.90	360
4-19	5200	1.86	4920	0.53	6,690,200	0.57	6,690,200
4-20	5630	1.86	4520	0.45	.....	0.49	.....
4-24	4600	1.86	5500	0.60	135,100	0.68	4,822,400
4-25	4280	1.86	5700	0.63	1,000	0.72	1,097,300
4-26*	4680	1.86	5500	0.59	97,500	0.67	1,493,600
4-27	5120	1.86	5750	0.61	700	0.67	1,250,400
4-28	5430	1.86	6250	0.65	500	0.70	578,800
4-29	5360	1.86	6000	0.62	1,207,600	0.69	1,207,600
1-5	2890	2.89	5500	0.61	63,400	0.63	1,027,200
2-5	2860	2.89	5000	0.56	976,100	0.58	976,100
3-5	3630	2.89	6000	0.60	418,600	0.64	467,200
5-1	6750	2.89	6500	0.53	.....	0.54	.....
5-3	4730	2.89	7500	0.68	1,500	0.72	23,200
5-4	5360	2.89	8000	0.70	1,700	0.76	1,700
5-5	4670	2.89	7400	0.68	1,300	0.72	402,900
5-6	5040	2.89	6500	0.58	71,200	0.61	15,871,700
5-3	5000	2.89	7000	0.62	202,800	0.66	11,217,700
5-10	2160	2.89	7000	0.90	100	0.91	100
5-11	3590	2.89	6000	0.61	39,800	0.64	39,800
5-12	4060	2.89	8000	0.77	530	0.81	530
5-13	4440	2.89	6000	0.56	1,133,100	0.59	3,666,500
5-14	5340	2.89	7000	0.61	13,000	0.64	13,000
5-15	4810	2.89	6000	0.54	239,000	0.57	239,000
5-16	4000	2.89	6500	0.63	2,100	0.67	4,300
5-17	5190	2.89	6500	0.57	87,800	0.60	87,800
5-18	Broke on first loading						
5-19	Broke on first loading						
5-20	4740	2.89	8880	0.81	170	0.85	900

*Reinforcing bar broke in pure moment region.

forcement and weak in shear was developed and found to fit both Moody's and the authors' data for 53 beams within ± 17 percent with a standard deviation of 0.095. The formula was developed by a combination of dimensional analysis and step-by-step numerical approximations as explained in Appendix 2. The expression developed is as follows:

$$V_c/bd = 1.90 f_c'^{0.53} (1 + 0.00562p^{1.8}) (1 + 0.685d^{1.42}/a^{1.42}) (1 + 13.4/d^{2.61}) \ldots . \text{(i)}$$

The ratio V_c/bd given by Eq. (i) can also be evaluated semigraphically as the product of the four factors obtained from Fig. 1 for the appropriate values of strength, steel percentage, span-depth ratio, and effective depth of the beam. More conveniently, the nomograph given in Fig. 2 may be used for the solution of Eq. (i).*

DEVELOPMENT OF THE SHEAR-MOMENT EXPRESSION

The maximum moments for the authors' beams which first cracked diagonally and then failed by destruction of the compression zone were compared with values given by Moody's and Laupa's formulas, and about one-

*For use of nomograph of Fig. 2, see Appendix 3.

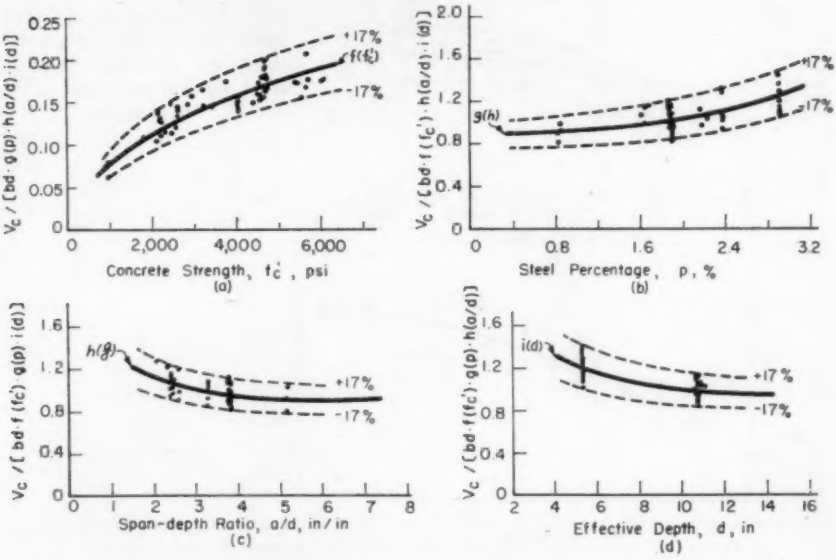

Fig. 1—Functional dependence of the static cracking load on concrete strength, steel percentage, span-depth ratio, and effective depth

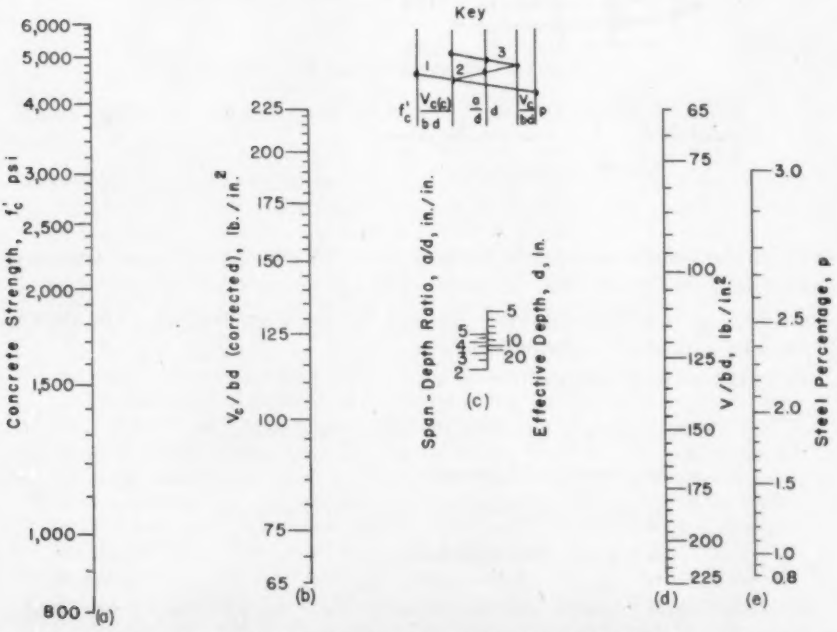

Fig. 2—Nomograph for predicting the static cracking load of simply supported reinforced concrete beams without web and compressive reinforcement

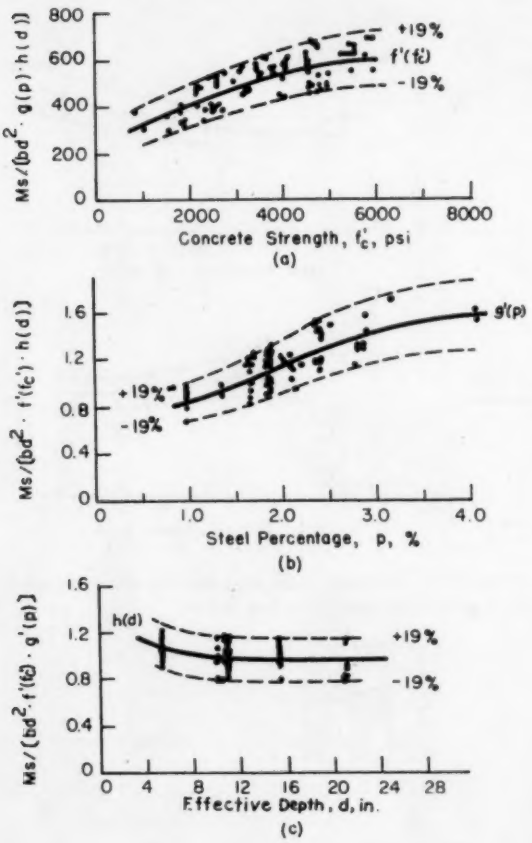

Fig. 3—Functional dependence of the static shear-moment on concrete strength, steel percentage, and effective depth

third of the results were found to vary by ± 35 percent. A new formula for determining the maximum shear-moment of beams with tensile reinforcement was therefore derived, again through dimensional analysis. The detailed procedure is given in Appendix 4.

The expression developed is:

$$\left.\begin{aligned} &M_s/bd^2 = 21.3 f_c'^{0.35} (1 + 0.188 p^{1.62}) (1 + 17.8/d^{3.25}) \\ &\text{for } p \text{ less than 2 5 percent} \\ &M_s/bd^2 = 12.8 f_c'^{0.35} (1 + 1.25\, p^{0.5}) (1 + 17.8\, d^{3.25}) \\ &\text{for } p \text{ greater than 2.5 percent} \end{aligned}\right\} \quad \text{(j)}$$

This expression predicts ultimate shear-moment for all existing data within ± 19 percent with a standard deviation of 0.102.

Eq. (j) can also be solved semigraphically as the product of the three factors

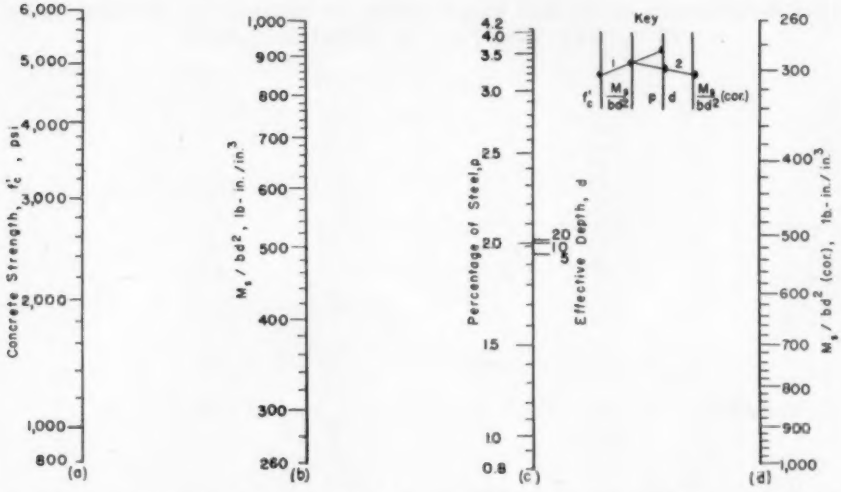

Fig. 4—Nomograph for predicting the static shear-moment of simply supported reinforced concrete beams without web and compressive reinforcement

given in Fig. 3. The nomograph in Fig. 4 may also be used to solve this equation.*

FAILURE OF BEAMS IN FATIGUE

Three modes of failures were observed in the 39 beams tested under repeated loading.

1. *Destruction of the compression zone* (*Fig. 5a, b, c, and d*)—Twenty-three of these beams failed by destruction of the compression zone at the end of the diagonal tension crack. They first cracked diagonally in either one or both of the shear spans. For simply supported beams this shear span is defined as being the length of the specimen between a support and the nearest load point. The diagonal cracks then extended toward the edges of the loading blocks accompanied by some splitting action along the reinforcement. The splitting in some cases proceeded to the ends of the beams (Fig. 5b) but always after the beams had "failed"; therefore, this splitting behavior was considered as a secondary failure. Final failures occurred when the compression zone at the top end of the diagonal cracks became too small to resist the applied load. The concrete in the compression zone was probably weakened by the repeated loading.
2. *Diagonal cracking failure* (*Fig. 5e*)—Eleven beams failed by "diagonal cracking." Generally, the diagonal cracks for these beams were fully developed as soon as the cracks were observed. The concrete failed by compression at the top of the loading block as soon as the cracks were formed. This means that the number of cycles for final failure of the beams was the same as that for cracking.
3. *Fatigue of reinforcement* (*Fig. 5f*)—Two beams failed by fatigue failure of the reinforcement in the region of pure flexure between the load points. The beams behaved similarly to those that failed by destruction of the compression zone; but after these beams cracked diagonally, the reinforcement in the beams failed through fatigue.

*For use of nomograph of Fig. 4, see Appendix 5.

TABLE 5—DAMAGE TESTS AND COMPARISON OF RESULTS OF BROKEN BEAMS PREVIOUSLY SUBJECTED TO REPEATED LOADING

Beam No.	f'_c, psi	p, percent	P_c (meas.), lb	P_s (meas.), lb	$\frac{P_c \text{ (meas.)}}{P_c \text{ (calc.)}}$	$\frac{P_s \text{ (meas.)}}{P_s \text{ (calc.)}}$
4-3	4580	1.86	.....	9,070	.....	1.12
4-6	4250	1.86	.....	8,170	.....	1.04
4-7	4650	1.86	.....	9,800	.....	1.19
4-8	4640	1.86	.....	6,620	.....	0.81
4-9	3910	1.86	.....	8,600	.....	1.13
4-11	3950	1.86	.....	8,010	.....	1.05
4-12	4290	1.86	.....	8,200	.....	1.04
4-15	2150	1.86	.....	6,890	.....	1.14
4-17	4300	1.86	.....	7,960	.....	1.01
4-19	5200	1.86	9,300	.....	1.00	....
4-20	5630	1.86	9,720	10,300	0.99	1.15
4-24	4600	1.86	10,470	.....	1.14	....
4-25	4280	1.86	.....	9,160	....	1.16
4-28	5430	1.86	.....	9,580	....	1.08
4-29	5360	1.86	10,970	.....	1.13	....
1-5	2890	2.89	.....	7,420	....	0.85
2-5	2860	2.89	8,000	8,320	0.89	0.97
3-5	3630	2.89	.....	9,700	....	1.03
5-1	6750	2.89	13,290	.....	1.09	....
5-3	4730	2.89	.....	12,250	....	1.18
5-4	5360	2.89	.....	9,250	....	0.88
5-5	4670	2.89	.....	9,980	.....	0.97
5-6	5040	2.89	.....	10,300	....	0.96
5-8	5000	2.89	.....	8,600	....	0.81
5-11	3590	2.89	9,100	9,150	0.93	0.98
5-12	4060	2.89	9,170	9,310	0.88	0.95
5-13	4440	2.89	.....	10,250	....	1.01
5-14	5340	2.89	.....	8,930	....	0.81
5-15	4810	2.89	10,030	10,610	0.90	1.02
5-17	5190	2.89	11,770	.....	1.03	....
5-19	3950	2.89	8,770	19,000	0.87	0.92
5-20	4740	2.89	.....	11,750	....	0.91

STUDY OF FATIGUE FAILURES

It has been observed that a reinforced concrete beam will fail at a load considerably less than its static load capacity if the load is repeated a sufficient number of times. One purpose of this investigation was to study statistically the fatigue life for initial diagonal tension cracking and for final failure of a beam which fails in shear when subjected to repeated loading varying from near zero to a maximum load. The span-depth ratio of all beams tested in fatigue was $a/d = 3.53$ or $a'/d = 3.72$.* Two values of steel percentages were used in this investigation, 1.86 and 2.89 percent, and the concrete strength varied from 2500 to 6500 psi. A minimum load of about 200 lb was maintained to prevent impact on the specimen during each cycle of loading. The maximum load varied about 4100 to 9500 lb, which is about 45 to 91 percent of the initial diagonal tension cracking load and about 51 to 100 percent of the static shear-moment capacity as predicted by Eq. (i) and (j).

Statistical studies of fatigue tests on concrete specimens are commonly based on plots of applied load, expressed in percent of the static load, against the logarithm of the fatigue life of the specimens.

Before any statistical study can be made, however, the criteria for failure of the beams must be determined. Comparison of Eq. (i) and (j) as plotted

*The distance a is measured from the edge of the loading block to the center of the support and is used to determine the shear-moments of the beams, while a' is measured from the center of the loading block to the center of the support and is used for the calculation of the initial diagonal tension cracking load of the beams. In these tests $a' = a + 1$ in.

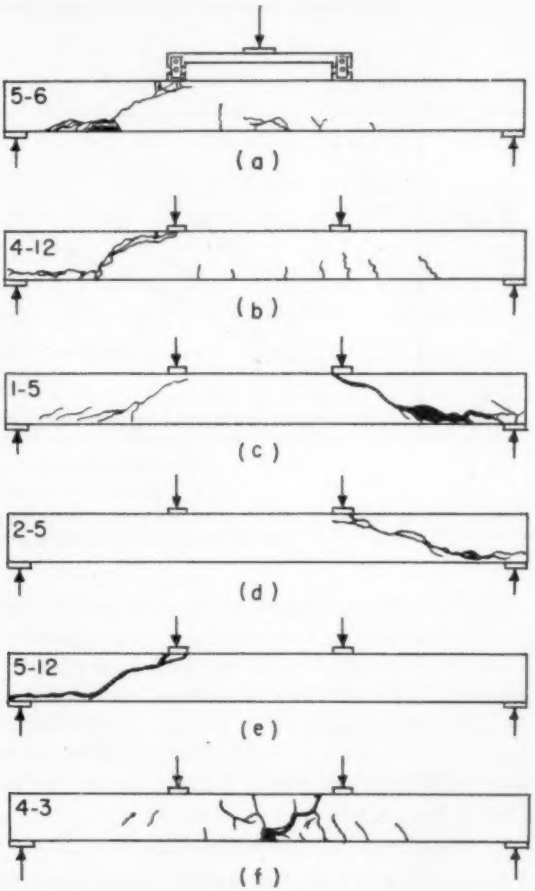

Fig. 5—Typical fatigue failures

in Fig. 6 indicates that the initial diagonal tension cracking load will be higher than the shear-moment load for beams under static loading with steel percentages of 1.86 and 2.89, when the concrete strength is between 2000 and 7000 psi. However, further consideration of the curves in Fig. 6 shows that the maximum variation between the cracking and shear-moment load is only 12 percent. Hence a variation of 6 percent of the actual loads from the theoretical loads could give a higher shear-moment load. This means that beams with this particular span-depth ratio and steel percentage when tested statically may fail either by diagonal tension or by destruction of the compression zone after a diagonal tension crack is fully developed. When loaded in fatigue, however, a diagonal tension crack may develop at a load considerably less than the initial diagonal tension cracking load predicted by Eq. (i), and failure may therefore occur in shear-compression. The fatigue life was studied for both initial diagonal cracking and final failure of the beams.

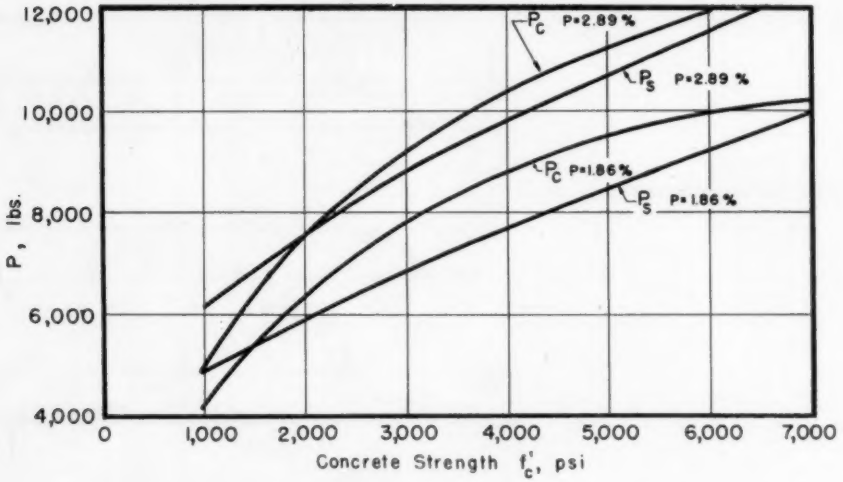

Fig. 6—Comparison of static load carrying capacities of beams with respect to concrete strength and steel percentage

The two beams that failed by fatigue of the reinforcing steel were reinforced with two #4 bars and had a steel percentage of 1.86. The beams all cracked diagonally before the bars failed in the region of constant moment. The average maximum stresses in the steel as determined from electric resistance strain gages were 31,000 and 32,600 psi for the two beams. The numbers of cycles of loading sustained by these beams were comparable to those of the beams which did not fail in the steel. Both had a fatigue life greater than one million cycles.

STATISTICAL STUDY OF INITIAL DIAGONAL TENSION CRACKING UNDER REPEATED LOADS

All beams that cracked diagonally in fatigue were included in the analysis. Two values of steel percentages, 1.86 and 2.89 percent, are available for analysis. The concrete strength of the beams varied from 2150 to 6750 psi. Statistical studies were made by plotting the applied maximum load in terms of percent of the static initial diagonal tension cracking load, as predicted by Eq. (i) against the logarithm of the number of cycles of loading the beams withstood before cracking diagonally. Separate studies for different steel percentages and concrete strength were made. Curves were drawn for a 50 percent probability of cracking and the "fatigue factor"* with a probability of 50 percent was determined as 0.57 for $p = 2.89$ percent, 0.57 for $p = 1.86$ percent, 0.58 for $f_c' \geqq 4300$ psi, and 0.57 for $f_c' \leqq 4700$ psi.

*Fatigue factor is defined as that ratio of the applied maximum repeated load to the static load for cracking or failure below which the beam will crack or fail at a given probability. The most common value used is the average fatigue factor which is the fatigue factor below which 50 percent of the beams tested will crack or fail.

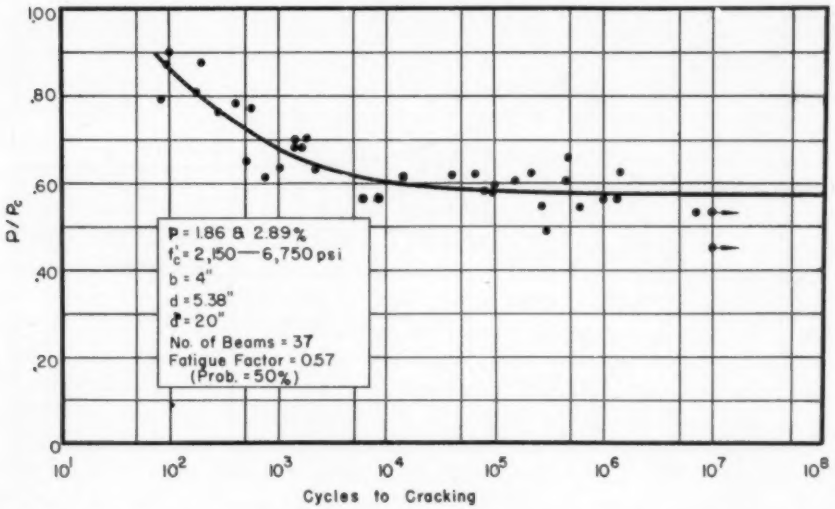

Fig. 7—Statistical study of the fatigue strength of beams against diagonal cracking

Since all the curves seem to have the same characteristics and all the fatigue factors seem to be about the same, a combined plot, Fig. 7, of beams with all steel percentages and concrete strengths was made. The data were found to lie within a limit of ± 15 percent from the 50 percent probability curve for cracking with one exception which was 17 percent off the curve. The fatigue factor for initial diagonal tension cracking with a probability of 50 percent is about 0.57 and with a probability of near zero percent is about 0.49.

STATISTICAL STUDY OF FINAL FAILURE UNDER REPEATED LOADS

All data for beams that failed under fatigue loading after a diagonal tension crack formed are included in this study. There are again two values of steel percentages, 1.86 and 2.89. The concrete strengths varied from 2150 to 6750 psi. Separate studies for final failures were again made for different concrete strengths and steel percentages by plotting the applied maximum load in terms of percent of the static shear-moment load, as expressed by Eq. (j), against the logarithm of the number of cycles of repeated load before failure. The failure curve and the fatigue factors with a probability of 50 percent were again found. The fatigue factors were 0.63 for $p = 2.89$ percent, 0.63 for $p = 1.86$ percent, 0.63 for $f_c' \geqq 4300$ psi, and 0.63 for $f_c' \leqq 4700$ psi.

Again, all the failure curves and fatigue factors were found to be very close to each other and therefore a combined study (Fig. 8) was made for all the beams. The failure curve with a probability of 50 percent was capable of predicting the results for all the beams tested within ± 15 percent, which is

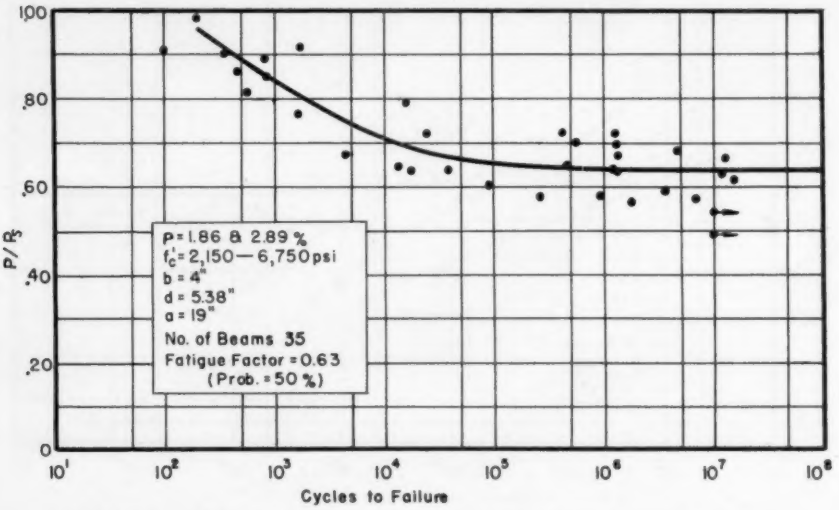

Fig. 8—Statistical study of the fatigue strength of beams against failure

within the range of variation for the prediction of static strength. The fatigue factor for final failure with a probability of 50 percent was found to be 0.63, and with a probability of near zero percent was about 0.56.

Since it was believed that the compression zone of a beam was not damaged by repeated loading before diagonal cracking occurred, a statistical study was made on the basis of the fatigue life as represented by the number of cycles of loading required to cause failure after diagonal cracking occurred. The results of this study are shown in Fig. 9. Excluding points that have relatively short lives after cracking, the data again seem to form a close band indicating the fatigue factor of 62 percent with a probability of 50 percent.

Due to the large flat portion of the curve, the fatigue life of a beam after cracking is more or less indeterminate, while the fatigue strength of the beam is quite certain. The fatigue factor of the beams can equally well be obtained by making statistical studies of the total fatigue lives of the beams. This study does tend to point out one important point, i.e., not much damage occurs in the shear-compression zone due to fatigue loading for a beam which has no diagonal cracks.

DAMAGE DUE TO REPEATED LOADING

Two beams, 4-20 and 5-1, did not crack diagonally and consequently did not fail even though 10,000,000 cycles of load were applied. They were removed from the fatigue machine and tested to failure under static load. The initial diagonal tension cracking load (Table 5) for both beams was found to be within the range of the formula derived by the authors. Beam 5-1 failed by diagonal tension cracking and destruction of the compression

zone simultaneously, and Beam 4-20 failed by destruction of the compression zone at a load slightly higher than the diagonal tension cracking load. The final failure load for Beam 4-20 (Table 5) was found to lie within the range of accuracy of the shear-moment expression derived in this paper.

Further static tests were made on some of the beams that failed in fatigue. Failure of a beam under repeated load generally occurred at only one end while the other end remained intact except for a few small tensile cracks or a diagonal tension crack. This end was then loaded at the center while simply supported over a span of 40 in. The results in Table 5 show that for the ends that did not crack diagonally in the fatigue tests, failure occurred either simultaneously by diagonal cracking and final destruction of the compression zone of the beams or by the destruction of the compression zone after the diagonal cracking load had been reached. In either case, the data obtained checked well with the authors' formulas. For ends that did have diagonal tension cracks as a result of the fatigue tests, failure was by destruction of the compression zone, and the recorded loads were consistent with the authors' formula for predicting the shear-moment capacity of the beams.

These test data seem to indicate two important results. First, if a beam did not crack diagonally under repeated loading, it was not damaged by the repeated loading so far as the initial diagonal tension cracking load and the shear-moment were concerned. Second, if the beam was cracked diagonally under repeated loading, the beam was not damaged so far as the shear-moment carrying capacity of the beam was concerned. It should be noted, however, that if the end of the beam had a diagonal crack, the ultimate load

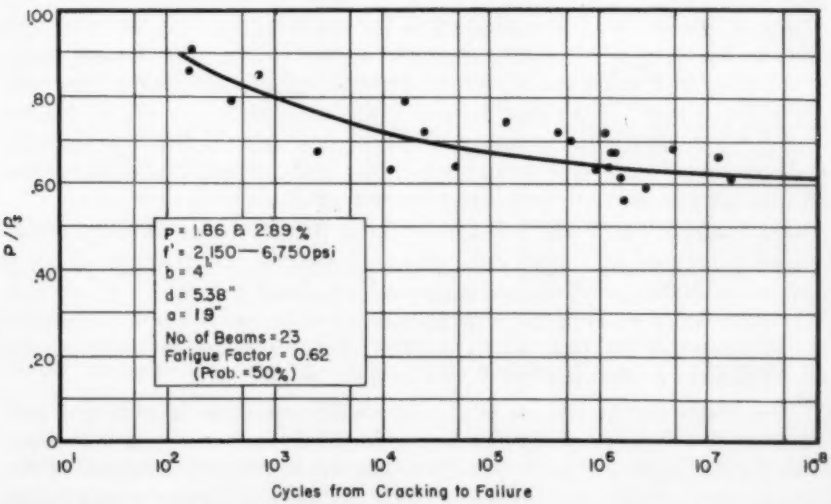

Fig. 9—Statistical study of the fatigue lives of beams against failure after cracking

would be predicted by the shear-compression theory, even if theoretically for static loading the beam might have a higher initial diagonal tension cracking load than the shear-moment load.

SUMMARY

The formulas developed by previous investigators for predicting the static diagonal tension cracking loads and shear-moments of simply supported reinforced concrete beams with tensile reinforcement only were found to be inapplicable for beams of relatively small cross section. Consequently, equations were developed using dimensional analysis to predict these loads, both for the relatively small beams tested by the authors and for similar beams of larger cross section for which adequate data could be found. The expression for initial diagonal tension cracking load, Eq. (i), was found to predict Moody's and the authors' test data for 53 beams within ± 17 percent with a standard deviation of 0.095. The expression for shear-moment, Eq. (j), was found to predict the existing data within ± 19 percent with a standard deviation of 0.102. Because of the complicated form of these two equations, nomographs were developed from which solutions may easily be obtained. Eq. (i) and (j) and the nomographs, Fig. 2 and 4, apply exclusively to simply supported reinforced concrete beams with only tensile reinforcement.

The repeated load tests indicate that the fatigue strength of the type of specimens tested is influenced by the percentage of steel and the concrete strength to the same extent as the static strength. When all the data, regardless of percentage of steel or concrete strength, for initial diagonal tension cracking in fatigue were plotted in Fig. 7 with the ordinate being the percentage of the cracking load applied as predicted by Eq. (i), the maximum variation was only ± 15 percent from the average curve. The fatigue factor for cracking with a probability of 50 percent is 0.57 and with a near zero probability is 0.49. A similar plot for final failure in shear, Fig. 8, yielded results with a maximum variation of only ± 15 percent from the average failure curve. The fatigue factors for failure with probabilities of 50 percent and near zero percent are 0.63 and 0.56, respectively.

Comparison of Fig. 7 and 8 indicates, up to 100,000 cycles, that the cracking load is reduced at a more rapid rate than the ultimate failure load. For example, at 1000 cycles the cracking load has been reduced to 68 percent of the static value while at the same life the ultimate load has been reduced to only 83 percent of the static ultimate. The fatigue factors at 10 million cycles and 50 percent probability are 0.57 and 0.63, respectively.

A few static tests made on beams previously subjected to repeated load indicated that if a beam did not crack diagonally under repeated loading, it was not damaged as regards either the diagonal tension cracking load or the ultimate moment carrying capacity. This finding was further strengthened by the statistical studies of fatigue lives of beams after cracking (Fig. 9).

Other tests indicated that if a beam was cracked diagonally in the fatigue test but did not fail, its static load-carrying capacity was not affected.

ACKNOWLEDGMENT

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APPENDIX 1—NOTATION

a = shear span of a beam measured from edge of loading block to center of support
a' = shear span of a beam measured from center of loading block to center of support
a_c = cracking span as defined by Morrow
A_s = area of tensile steel of a reinforced concrete beam
b = width of a rectangular reinforced concrete beam
c = constant
d = effective depth of a reinforced concrete beam
d' = effective depth of a beam which needs no size correction for computing initial diagonal tension cracking load or shear-moment of the beam
E_c = modulus of elasticity of concrete
E_s = modulus of elasticity of steel
f_r = modulus of rupture of concrete
f_s = stress in tensile reinforcement
f_c' = ultimate strength of concrete as determined from 6 x 12-in. cylinders
f'_{up} = yield point of web reinforcement
jd = moment arm between the resultant compressive force in the concrete and the tensile stress in the steel as computed by the straight line formula
K = constant relating to the contribution of web reinforcement by truss analogy
k = ratio of the distance between the top of the beam and the neutral axis of the beam to the effective depth of the beam as computed from the straight line theory for flexure
k_1k_3 = ratio of average concrete stress in the compression zone to the strength of 6 x 12-in. cylinders
k_2 = ratio of distance from top fiber to center of compression to distance from top fiber to neutral axis
M = moment
M_s = shear-moment
n = ratio of the modulus of elasticity of steel to that of concrete
p = percentage of tensile reinforcement
P = total load on a simply supported beam
P_c = diagonal cracking load
P_s = total load on a simply supported beam causing a maximum moment equal to the shear-moment
r = percentage of web reinforcement
V = shear force
v = nominal unit shear stress
V_c = shear force for initial diagonal tension cracking
V_s = shear force at which the shear-moment of the beam is developed
V_u = shear force at failure of the beam
v_u = nominal unit shear stress at failure

APPENDIX 2—DEVELOPMENT OF EQUATION FOR INITIAL DIAGONAL TENSION CRACKING LOAD

The following is the procedure used to develop the expression for initial diagonal tension cracking load.

Experience has indicated that the main variables that influence the initial diagonal tension cracking load of a concrete beam with tensile reinforcement are concrete strength, percentage of tensile reinforcement, effective depth of the beam, shear span, width of the beam, modulus of elasticity of the concrete, modulus of elasticity of the tensile steel, and relative size of the beam. Therefore the following expression can be written:

$$V_c = F_1 (f_c', p, d, a, b, E_c, E_s, \text{relative size})$$

$$\text{or } F_2 (V_c, f_c', p, d, a, b, E_c, E_s, \text{relative size}) = 0 \quad \ldots\ldots\ldots\ldots\ldots\ldots (k)$$

If the assumption is made that the modulus of elasticity of concrete is a function of the concrete strength and that the term, relative size, can be expressed by the ratio of the effective depth of the beam to a standard effective depth for which correction of size is not necessary, then the following expression results:

$$F_3 (V_c, f_c', p, d, a, b, E_s, d/d') = 0$$

$$\text{or } F_4 (V_c, f_c', p, d, a, b, E_s, d') = 0 \quad \text{.....(l)}$$

Dimensional analysis

The purpose of this analysis is to reduce the eight variables that appear in Eq. (l) to a function that contains a minimum number of dimensionless products forming a complete set, or the minimum number still forming a function which describes the phenomenon completely.

By forming a dimensional matrix for Eq. (l) as shown in Table 6a, it is apparent that the determinant formed by the last two columns of the matrix is not equal to zero. This means

TABLE 6a—DIMENSIONAL MATRIX FOR THE FUNCTION OF THE SHEAR FORCE FOR INITIAL DIAGONAL TENSION CRACKING

	V_c	f_c'	p	d	a	b	E_s	d'
F	1	1	0	0	0	0	1	0
L	0	−2	0	1	1	1	−2	1

$$\begin{vmatrix} 1 & 0 \\ -2 & 1 \end{vmatrix} = 1 \neq 0$$

TABLE 6b—SOLUTION MATRIX FOR THE FUNCTION OF THE SHEAR FORCE FOR INITIAL DIAGONAL TENSION CRACKING

	V_c	f_c'	p	d	a	b	E_s	d'
$V_c/E_s d^2$	1	0	0	−2	0	0	−1	0
p	0	0	1	0	0	0	0	0
b/d	0	0	0	−1	0	1	0	0
a/d	0	0	0	−1	1	0	0	0
d/d'	0	0	0	1	0	0	0	−1
f_c'/E_s	0	1	0	0	0	0	−1	0

$$\begin{vmatrix} 0 & -2 & 0 & 0 & -1 & 0 \\ 1 & 0 & 0 & 0 & 0 & 0 \\ 0 & -1 & 0 & 1 & 0 & 0 \\ 0 & -1 & 1 & 0 & 0 & 0 \\ 0 & 1 & 0 & 0 & 0 & -1 \\ 0 & 0 & 0 & 0 & -1 & 0 \end{vmatrix} = 2 \neq 0$$

that the dimensional matrix is of the rank of two, and therefore the number of dimensionless products necessary to form a complete set, which will describe the function $F_4 = 0$, is eight minus two or six.

By observation,* $V_c/E_s d^2$, p, b/d, a/d, d/d', and f_c'/E_s are indicated as the six dimensionless products. To show that they form a complete set, it is necessary to show the solution matrix formed by these six dimensionless products is of the rank of six. This is indeed the case when the matrix is written as shown in Table 6b, since the determinant formed by the last six columns of the matrix is not zero.

The foregoing analysis indicates that the equation of eight variables can be replaced by a function of six dimensionless products as shown in Eq. (m):

$$F_5 (V_c/E_s d^2, p, b/d, a/d, d/d', f_c'/E_s) = 0$$

$$\text{or } V_c/E_s d^2 = F_6 (p, b/d, a/d, d/d', f_c'/E_s) \quad \text{.....(m)}$$

If it is assumed that V_c is proportional to b, then Eq. (m) becomes:

$$V_c/E_s d^2 = b/d \quad F_7 (f_c', p, a/d, d/d')$$

$$\text{or } V_c/E_s bd = F_7 (f_c', p, a/d, d/d') \quad \text{.....(n)}$$

But since the modulus of elasticity of steel can be considered constant, as is also the standard effective depth, the equations further reduce to:

$$V_c/bd = F_8 (f_c', p, a/d, d) \quad \text{.....(o)}$$

*There are standard methods to obtain a complete set of dimensionless products but these are not necessary here.

It can be shown that F_s can be represented by the product of four functions $f(f_c')$, $g(p)$, $h(a/d)$, and $i(d)$,* thus:

$$V_c/bd = f(f_c') \cdot g(p) \cdot h(a/d) \cdot i(d) \ldots\ldots\ldots\ldots\ldots\ldots (p)$$

This is a simple expression. The only step that remains now is to find these functions for the experimental data available to the authors. To do this, a numerical method of successive approximation was developed.

Numerical analysis

First, the initial diagonal tension cracking load of beams with the same steel percentage, span-depth ratio, and effective depth were plotted against the concrete strength. A tentative function of concrete strength, $f(f_c')$ in Eq. (p), was developed. Then, the initial diagonal tension cracking load of beams with the same span-depth ratio and effective depth were divided by this function according to their corresponding concrete strengths, and were plotted against the steel percentage. A tentative function of steel percentage, $g(p)$ in Eq. (p) was then determined. All the initial diagonal tension cracking loads for beams with the same effective depth were then divided through by the functions of concrete strength and steel percentage and plotted against the span-depth ratio, and a tentative function of the span-depth ratio resulted. Then, dividing all initial diagonal tension cracking loads available by the functions of concrete strength, steel percentage, and span-depth ratio just determined, and plotting the values obtained against the corresponding effective depth of the beams, a tentative function of the effective depth is obtained. Using these tentative functions of the steel percentage, span-depth ratio, and the effective depth, an improved function of concrete strength was then found. Continuing the cyclic numerical evaluation of these four functions the results converged to:

$$V_c/bd = 1.90 f_c'^{0.53} (1 + 0.00562\, p^{1.8}) (1 + 0.685\, d^{1.42}/a^{1.42}) (1 + 13.4/d^{2.61}) \ldots (i)$$

All the final functions of concrete strength, steel percentage, span-depth ratio, and effective depth are shown graphically in Fig. 1.

APPENDIX 3—USE OF NOMOGRAPH TO COMPUTE DIAGONAL TENSION CRACKING LOAD

The following is an outline of the procedure for obtaining the initial diagonal tension cracking load of a beam simply supported without web and compressive reinforcement by using the nomograph in Fig. 2.

1. A straight line is passed through the values of concrete strength and steel percentage of the beam on vertical lines (a) and (e) of the nomograph. The intercept of this straight line with (b) is then marked.
2. Connecting this intercept with the value of the span-depth ratio of the beam on the left side of the line (c), a value of unit shear force for initial diagonal tension cracking is read on line (d). This unit shear force for diagonal tension cracking is not corrected for size effect.
3. For a more correct value for the unit shear force for initial diagonal tension cracking, connect the uncorrected value with the value of the effective depth of the beam on the right side of line (e), and obtain the corrected value of the unit shear force from line (b), the V_c/bd (corrected) line.

*See explanation in Appendix 6.

4. The shear force for initial diagonal tension cracking is obtained by multiplying the final value by the width and the effective depth of the beam. For beams loaded at the third points or at the center, the diagonal tension cracking load for the beam is twice the shear force for initial diagonal tension cracking.

APPENDIX 4—DEVELOPMENT OF EQUATION FOR SHEAR-MOMENT

The following method was used to determine the expression for the magnitude of the shear-moment.

Variables

If the theory of shear-compression failure is correct,* then the variables that will influence the shear-moment of a beam with tensile reinforcement are k, f_c', p, n, b, d, and E_s or k_1k_2, f_c', p, f_s, b, d, and E_s as suggested by Laupa's and Moody's formulas, respectively. A closer examination of either set of these variables shows that the actual independent variables are M_s, f_c', p, n, b, d, and E_s, or

$$F_1'(M_s, f_c', p, n, b, d, E_s) = 0 \quad \text{.....(q)}$$

By introducing the size effect through a ratio of the effective depth of the beams to a standard effective depth, the function becomes:

$$F_2'(M_s, f_c', p, b, d, d', E_s) = 0 \quad \text{.....(r)}$$

Dimensional analysis

The function of seven variables as appeared in Eq. (r) was further reduced to a function of a complete set of five dimensionless products through dimensional analysis. The analysis is essentially the same as that for the formula for initial diagonal tension cracking and will not be shown here. The resulting function after the analysis is:

$$F_3'(M_s/E_sd^3, p, b/d, f_c'/E_s, d/d') = 0$$

$$\text{or } M_s/E_sd^3 = F_4'(p, b/d, f_c'/E_s, d/d') \quad \text{.....(s)}$$

If M_s is assumed to be directly proportional to b, then,

$$M_s/E_sd^3 = b/d \cdot F_5'(p, f_c'/E_s, d/d')$$

$$\text{or } M_s/E_sbd^2 = F_5'(p, f_c'/E_s, d/d') \quad \text{.....(t)}$$

But E_s can be considered constant and d' is a constant, therefore Eq. (t) further reduces to:

$$M_s/bd^2 = F_6'(f_c', p, d)$$

$$\text{or } M_s/bd^2 = f'(f_c') \cdot g'(p) \cdot h'(d) \quad \text{.....(u)}$$

*With deformed bars meeting ASTM A 305 or end anchorages, bond failures are rare and diagonal cracking will occur before reaching the critical shear-moment if the span-depth ratio of the beam is reasonably small, but not so small that the shear force will have a large effect on the compressive force in the compression zone.

The functions $f'(f_c')$, $g'(p)$, and $h'(d)$, are again found through numerical approximation. The functions are shown graphically in Fig. 3. The final expression for the shear-moment of a concrete beam of any size with tensile reinforcements is:

$$\left.\begin{aligned} M_s/bd^2 &= 21.3\, f_c'^{0.35}\,(1 + 0.188\, p^{1.62})\,(1 + 17.8/d^{3.25}) \\ &\text{for } p \text{ less than 2.5 percent} \\ M_s/bd^2 &= 12.8\, f_c'^{0.35}\,(1 + 1.25\, p^{0.5})\,(1 + 17.8/d^{3.25}) \\ &\text{for } p \text{ greater than 2.5 percent} \end{aligned}\right\} \quad \ldots\ldots\ldots\ (j)$$

APPENDIX 5—USE OF NOMOGRAPH TO COMPUTE SHEAR-MOMENT

The following is an outline of the procedure for obtaining the shear-moment of a concrete beam without web and compressive reinforcement by using the nomograph in Fig. 4.

1. A straight line is passed through the values of concrete strength and steel percentage of the beam on vertical lines (a) and (c) on the nomograph. An intercept of this straight line on (b) gives a unit shear-moment for the beam, but which is not corrected for size effect.
2. A correction for size effect of this unit shear-moment can be made by connecting the uncorrected value and the value of the effective depth of the beam on the right side of vertical line (c), and obtaining the corrected value of the unit shear-moment of the beam on the vertical line (d).
3. The shear-moment of the beam is obtained by multiplying the unit shear-moment by the width, and the square of the effective depth of the beam.

APPENDIX 6—TRANSFORMATION OF FUNCTION OF DIMENSIONLESS PRODUCTS INTO A MULTIPLE OF FUNCTIONS

A function of a complete set of n dimensionless products can generally be expressed by a multiple of n functions, each of which varies with one of the n dimensionless products. This can be proved by noting that there are infinite numbers of sets of dimensionless products which can describe the physical phenomenon completely, or

$$F(\pi_1, \pi_2, \ldots \pi_n) = F^1(\pi_1', \pi_2', \ldots \pi_n') \ldots = F^i(\pi_1^i, \pi_2^i, \ldots \pi_n^i) = \ldots$$

In particular, the case of $n = 2$, then the problem reduces to the following:

$$\begin{aligned} F(\pi_1, \pi_2) &= f_1(\pi_1) \cdot F_2(\pi_2), \\ F_1(\pi_1', \pi_2') &= f_1'(\pi_1') \cdot f_2'(\pi_2'), \\ &\ldots\ldots\ldots\ldots\ldots\ldots, \text{ or} \\ F^i(\pi_1^i, \pi_2^i) &= f_1^i(\pi_1^i) \cdot f_2^i(\pi_2^i) \\ &\ldots\ldots\ldots\ldots\ldots\ldots \end{aligned}$$

It can be shown that the function $F(\pi_1, \pi_2)$ can be expressed as a sum of the multiple of functions which are functions of π_1 and π_2.

$$\begin{aligned} F(\pi_1, \pi_2) &= g_1(\pi_1) \cdot h_1(\pi_2) + g_2(\pi_1) \cdot h_2(\pi_2) + \ldots + \\ &= g_j(\pi_1) \cdot h_j(\pi_2) + \ldots \end{aligned}$$

where $g_j(\pi_1)$ and $h_j(\pi_2)$ are the following type: $a_j\pi_1^k$ and $b_j\pi_2^m$.

But $g_1(\pi_1)\,h_1(\pi_2) + g_2(\pi_1)\,h_2(\pi_2)$ can be expressed by a function of multiple of two new dimensionless products:

$$g_1(\pi_1)\,h_1(\pi_2) + g_2(\pi_1)\,h_2(\pi_2)$$

$$= g_1(\pi_1)\,h_1(\pi_2)\left[\frac{g_2(\pi_1)\,h_2(\pi_2)}{g_1(\pi_1)\,h_1(\pi_2)}\right] + 1$$

$$= g'(\pi_1')\,h'(\pi_2')$$

or by analogy,

$$F(\pi_1, \pi_2) = g_1(\pi_1)\,h_1(\pi_2) + \ldots + g_n(\pi_1)\,h_n(\pi_2)$$

$$= f_1'(\pi_1')\,f_2'(\pi_2')$$

where π_1' and π_2' can be proved to still form a complete set which describes the physical phenomenon completely. Since $F(\pi_1, \pi_2) = F^1(\pi_1^1, \pi_2^1)$, therefore,

$$F^1(\pi_1^1, \pi_2^1) = f_1^1(\pi_1^1)\,f_2^1(\pi_2^1)$$

This proof can be extended to the case of n dimensionless products so that the following is true:

$$F(\pi_1, \pi_2 \ldots \pi_n) = f_1(\pi_1)\,f_2(\pi_2) \ldots f_n(\pi_n)$$

Discussion of this paper should reach ACI headquarters in triplicate by Sept. 1, 1958, for publication in the Part 2, December 1958 JOURNAL.

Uniform Structural Lightweight Aggregate Concrete Through Careful Proportioning and Control*

By PAUL J. FLUSS†

SYNOPSIS

Variations in bulk specific gravity of the aggregate and construction requirements make it necessary to give special attention to the careful proportioning, proper control, and adjustment of structural lightweight aggregate concrete. Experience on the Ferry Building, San Francisco, is cited.

SPECIFICATIONS

Since the Ferry Building in San Francisco rests on piles, lightweight aggregate concrete was used to remodel its north wing for the World Trade Center.

The specifications asked for:

(1) Two classes of concrete:

Class	A	B
Cement content, sacks per cu yd	6.5	5.9
Maximum water-cement ratio, gal. per sack	6.75	7.25
Slump, in.	3–5	2–4
28-day compressive strength, psi	2500	2000
Maximum oven dry weight, lb per cu ft	100	100

(2) An expanded shale (Haydite), or an approved equal lightweight aggregate.

(3) A plasticizing and air-entraining admixture (1.50 lb per cu yd), or a plasticizing and water-reducing admixture (3.75 lb per cu yd), or any other approved admixture.

Upon request of the contractor the following were approved:

(1) For Class A concrete in locations where it was necessary for proper placing, a slump of 5-6 in. with a corresponding increase in cement content to 6.75 sacks.

(2) The use of a different expanded shale lightweight aggregate (Basalite), which due to its round shape gives better workability though for the same reason has a greater tendency to float to the top when the concrete is vibrated, especially in higher slump mixes.

It was decided to use the plasticizing and air-entraining admixture, to increase the workability and plasticity of the mix and consequently reduce the water requirements at the same slump.

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†Member American Concrete Institute, Testing Engineer, San Francisco Port Authority, San Francisco, Calif.

MIX PROPORTIONING

After preliminary mix calculations, a number of trial batches were made with varying percentages of lightweight aggregate sand and different structural sands (Table 1) and the following properties determined: unit weight, wet, hardened (1 day old), and oven dry; yield; cement content; slump; workability; air content; 7- and 28-day compressive strength.

TABLE 1—SAND GRADINGS

Sieve No.	Percent passing			
	Lightweight aggregate sand	Stream deposited aggregate		Windblown Pacific Coast dune sand
		Rodmill sand	Natural sand	
4	99	99	99	
8	74	95	79	
16	50	66	52	
30	32	38	27	100
50	19	19	11	96
100	8	8	4	10
200	2.3	4	1.5	0.5

TABLE 2—MIXES SELECTED FROM TRIAL BATCHES

Class	A	B
Lightweight aggregate, maximum size 1 in., lb	380	330
Lightweight aggregate, maximum size 9/16 in., lb	440	400
Lightweight aggregate sand, lb	540	580
Natural sand, lb	660	640
Dune sand, lb	200	190
Cement, lb	611 and 635	555
Admixture, lb	1	1
Unit weight, wet, lb per cu ft	111.65	107.25
Unit weight, hardened, lb per cu ft	112.30	108.30
Unit weight, oven dry, lb per cu ft	98.10	94.65
Yield, cu ft	27.25	27.00
Cement content, sacks per cu yd	6.59	5.9
Slump, in.	3½	3
Workability	Excellent	Very good
Air content, percent	4.2	—
7-day compressive strength, psi	1925	1910
28-day compressive strength, psi	3310	2700

The problem was to use the largest possible percentage of lightweight aggregate sand while getting the optimum workability without any sacrifice in compressive strength. This was achieved by using approximately 20 percent well rounded dune sand as part of the structural sand.

Table 2 shows the mixes for the two classes of concrete, selected from the trial batches, as starting points for actual production.

Preliminary practical test

The structural design called, in some places, for 6-in. walls, reinforced by a curtain of #4 bars at 12-in. centers. To find out whether the lightweight aggregate would segregate or float when vibrating Class A concrete, placed in 6-ft lifts, the contractor built a test wall. It was determined that only small diameter vibrators could be used without any detrimental effect. After curing, the test wall was cut and showed an excellent distribution of all sizes of lightweight aggregate.

CONTROL AND ADJUSTMENT

Before going into production, an expedient system of control and adjustment had to be devised to compensate for variations in bulk specific gravity of the lightweight aggregate due to unavoidable differences in (1) expansion and (2) saturation, although the material was kept wet constantly by sprinkling.

To keep up with production it was decided to base all figures on the wet unit weight and yield, using the actual batch weights. Several sets of batch

TABLE 3—CONCRETE PLANT REPORT ON LIGHTWEIGHT AGGREGATE CONCRETE

Load No.	1	2	3	4	5	6	7
Mix No.	3	2	5	1	1	2	2
Lightweight aggregate, 1 in. max., lb	350	360	370	380	380	360	360
Lightweight aggregate, 9/16 in. max., lb	410	420	430	440	440	420	420
Lightweight sand, lb	500	510	523	540	540	510	510
Concrete sand, natural, lb	660	660	660	660	660	660	660
Dune sand, lb	200	200	200	200	200	200	200
Cement, lb	611	611	611	611	611	611	611
Added water, lb	184	184	184	184	200	192	192
Total, lb	2916	2946	2979	3016	3032	2954	2954
Unit weight, wet, lb per cu ft	112.0	112.5	112.5	108.75	107.5	108.0	108.1
Yield, cu ft	26.0	26.2	26.5	27.7	28.2	27.35	27.3
Cement content, sacks per cu yd	6.75	6.70	6.62	6.35	6.22	6.42	6.43
Slump, in.							3½
Cylinder No.							108
Admixture, lb	1	1	1	1	1	1	1

tickets were provided in which only the lightweight aggregate weights were proportionately increased or decreased, while the added water was to be gaged to arrive at the desired slump, which was checked regularly. When the yield was off more than approximately ½ cu ft either way the batch weights were changed accordingly. Similarly the added water was increased or decreased when an adjustment of the slump was necessary. Whenever the ready-mixed concrete plant had to change over to conventional concrete—due to intermittent concreting—part of the mixing water was withheld and used to wash out the mixer. In this case the material had to receive an additional mixing in the truck before the unit weight sample was taken. Frequently it happened that the first trucks arrived at the job with concrete having too low a slump. This was probably due to the fact that the lightweight aggregate had not been fully saturated and absorbed part of the mixing water. Water was then added under the supervision of an inspector. For this reason the compression test cylinders and the corresponding slumps were taken on the job, after any necessary addition of water.

Table 3 shows the form used for the control and adjustment.

TABLE 4—TEST RESULTS

Class of concrete	A		A		B	
Sacks of cement per cu yd	6.75		6.5		5.9	
	Range	Average	Range	Average	Range	Average
Unit weight, wet, lb per cu ft	107.50–115.00	111.40	107.50–113.00	110.20	106.90–111.90	109.75
Unit weight, hardened, lb per cu ft	107.40–118.45	112.64	106.75–114.30	111.84	106.10–113.65	110.00
Unit weight, oven dry, lb per cu ft	Not enough data available		93.00–101.75	98.00	94.45– 99.50	96.3
Cement content, sacks per cu yd	6.53–7.00	6.74	6.36–6.60	6.47	5.63–6.00	5.91
Slump, in.	3 to 6¼	5¼	3 to 6	4½	3 to 6	4½
28-day compressive strength, psi	3050–3840	3473	2950–3930	3357	2860–3510	3170
Standard deviation, psi	236		244		185	
Coefficient of variation, percent	6.8		7.3		5.8	

Fig. 1—Samples of lightweight aggregate concrete from floor slab and wall

TEST RESULTS

The test results in Table 4 show that the unit weights and cement contents fluctuated due to the nature of the lightweight aggregate, while the slumps were adjusted on the job to suit particular requirements, which probably accounts for their wider variations. However, the 28-day compressive strengths and their coefficients of variation were well below the 10 percent established for an excellent control standard for general construction.

Distribution of lightweight aggregate

Though the test data bring out the structural strength of the concrete there was no proof that the material in place had not segregated, especially in the higher slump mixes which were used for the 6-in. walls.

When the World Trade Center was ready for occupancy many of the prospective tenants asked for alterations to fit their own needs. Among other things several windows had to be cut into the 6-in. east wall and ducts and conduits had to be accommodated in the floors. Fig. 1 shows that no segregation or floating up had taken place and that all sizes of lightweight aggregate are evenly distributed.

CONCLUSIONS

The results indicate that proper mix proportioning and, above all, constant as well as expedient control make it possible to overcome the variations inherent in lightweight aggregate and to produce structural concrete comparable in uniformity to that with conventional aggregate.

Discussion of this paper should reach ACI headquarters in triplicate by Sept. 1, 1958, for publication in the Part 2, December 1958 JOURNAL.

Effect of Mixing and Curing Temperature on Concrete Strength*

By PAUL KLIEGER†

SYNOPSIS

Comprehensive study was made of compressive and flexural strengths produced by different types of portland cement used in concretes mixed, placed, and cured at various temperatures between 25 F and 120 F. Tests indicate that there is a temperature during the early life of concrete which is considered optimum with regard to strength at later ages.

Effect of calcium chloride on strength at varying temperatures of mixing, placing, and curing is reported. Effect of cement temperature was found unimportant, except as it affected concrete temperature after mixing. More air-entraining agent was required for given air content as concrete temperature increased and slump decreased.

INTRODUCTION

This study was prompted by the need for more detailed information on the influence of temperature during mixing, placing, and curing on concrete strength development. The necessity for continuing concreting operations despite high temperatures during the summer and low temperatures during winter is increasing. Although it is known that high temperatures increase the rate of hydration of cement, the effect of this increase on strength development needs investigating.

Specific information was sought on compressive and flexural strengths produced by different types of portland cement when used in concretes mixed, placed, and cured at various temperatures ranging from near the freezing point to 120 F. Information on the effects of an accelerator under these different conditions was also desired.

Recent tests of the influence of cement temperature on concrete strength are reported also.

SCOPE OF TESTS

Part I of this study embraced temperatures from 73 F to 25 F; Part II covered temperatures from 73 F to 120 F. Mixing, placing, and curing of concretes were conducted at temperatures of 120, 105, 90, 73, 55, and 40 F. In addition, some concretes mixed and placed at 40 F were cured at 25 F.

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†Member American Concrete Institute, Senior Research Engineer, Applied Research Section, Portland Cement Association, Chicago, Ill.

TABLE 1—CHEMICAL COMPOSITION AND POTENTIAL COMPOUND COMPOSITION OF CEMENTS

Chemical analyses made in accordance with ASTM methods of test current in October, 1953, for Part I and June, 1954, for Part II. Sodium oxide and potassium oxide by flame photometry, ASTM C 228-49T. Compounds corrected for free CaO.

Cement		Major components, percent							Minor components, percent						Calculated compound composition, percent					
Lot No.	ASTM type	SiO_2	Al_2O_3	Fe_2O_3	Total CaO	MgO	SO_3	Ignition Loss	Mn_2O_3	Free CaO	Insoluble residue	Alkalies Na_2O	Alkalies K_2O	Alkalies Total as Na_2O	C_3S	C_2S	C_3A	C_4AF	$CaSO_4$	Free CaO
Part I, 73 F and below																				
18827*	I	21.20	5.93	2.64	63.12	2.46	2.12	1.26	0.29	0.90	0.12	0.22	0.67	0.66	42.5	28.8	11.3	8.0	3.61	0.90
19012	III	19.88	5.57	2.62	65.08	1.90	2.63	1.67	0.21	1.71	0.10	0.19	0.42	0.47	58.2	13.1	10.3	8.0	4.47	1.71
18830	II	23.13	4.45	3.36	62.50	2.01	1.75	2.01	0.41	1.22	0.14	0.14	0.21	0.28	33.9	40.7	6.1	10.2	2.98	1.22
Part II, 73 F and above																				
18922†	I	21.25	5.86	2.76	63.17	2.43	2.15	1.11	0.24	1.07	0.13	0.23	0.63	0.64	41.8	29.4	10.9	8.4	3.66	1.07
18931†	III	20.12	6.04	2.65	64.67	0.54	2.74	1.23	0.19	1.08	0.20	0.18	0.47	0.49	53.8	17.1	11.5	8.1	4.65	1.08
18925†	II	22.27	4.89	3.41	62.50	2.91	1.79	1.30	0.19	0.84	0.17	0.20	0.47	0.51	39.1	34.4	7.2	10.4	3.04	0.84

*Results for Lot 18827 are averages of tests of the four individual brands used to make this blend.
†Results are averages of tests on individual cements comprising each blend: 18922, four brands; 18931, two brands; 18925, two brands.

TABLE 2—PHYSICAL TESTS OF CEMENT AND MORTARS

Cement		Fineness			Normal consistency, percent	Time of setting, hr and min				Autoclave expansion, percent	Air content, percent, 1:4 mortar	Tensile strength, psi, 1:3 standard sand mortar briquets				Compressive strength, psi, 2-in. plastic mortar cubes			
						Vicat		Gillmore											
Lot No.	ASTM type	Specific surface, sq cm per g		Passing 325 mesh, percent		Initial	Final	Initial	Final			1 day	3 da	7 da	28 days	1day	3days	7 days	28days
		Wagner	Blaine																
Part I, 73 F and below																			
18827	I	1690	3310	90.4	25.0	4:00	6:55	4:30	7:10	0.13	9.7	205	365	410	515	790	1970	3150	4530
19012	III	2350	4840	98.4	29.0	1:45	4:00	2:55	4:20	0.16	6.3	360	465	540	580	2310	4030	5780	6850
18830	II	2020	3910	93.4	25.0	5:05	8:45	5:45	9:15	0.02	6.8	115	240	380	495	520	1180	1910	3730
Part II, 73 F and above																			
18922	I	1650	3120	89.6	24.5	3:00	—	3:55	6:15	0.11	8.5	195	365	400	450	780	1910	2910	4620
18931	III	2360	4730	98.1	28.5	2:00	4:00	2:35	4:30	0.10	6.6	360	485	505‡	530	2090	4010	5440‡	6780
18925	II	2010	3880	92.5	24.0	3:30	5:00	4:10	5:25	0.05	7.8	170	330	400‡	495	680	1510	2600‡	4550

*Tests made in accordance with ASTM Standards current in May, 1954. †Tests made in accordance with ASTM Standards current in June, 1954. ‡Average of two specimens.

ASTM Types I, II, and III cements were used. Concretes were made both with and without calcium chloride as an accelerator. An air-entraining agent was added at the mixer to entrain a prescribed amount of air in all of the concretes.

Concretes were tested at ages ranging from 1 day to 1 year. Included were both compressive and flexural strength tests. In a number of tests the temperature of cement at the time of mixing was varied to determine the influence on the strength of concretes produced.

MATERIALS

Cement

ASTM Types I, II, and III cements were used in this study. For both Parts I and II, the Type I cements were blends of four different brands and the Type III blends of two different brands. One brand of Type II was used in Part I, while a blend of two brands of Type II was used in Part II. Chemical analyses, calculated potential compound composition, and results of miscellaneous physical tests of these cements are shown in Table 1. The results of strength tests of mortars made with these cements are shown in Table 2.

Aggregates

Sand and gravel from Elgin, Ill., were used in the gradings shown in Table 3. These aggregates are partly siliceous and partly calcareous natural sand and gravel.

Admixtures

Neutralized Vinsol resin in solution, added at the mixer, was the air-entraining agent. The accelerator was commercial flake calcium chloride dissolved in a portion of mixing water prior to use.

MIXING, PLACING, AND CURING TEMPERATURES

Concretes were mixed and placed at temperatures of 40, 55, 73, 90, 105, and 120 F. Specimens were prepared for test at ages of 1, 3, 7, and 28 days, 3 months, and 1 year.

In Part I, all specimens were cured continuously moist (100 percent relative humidity) at the mixing and placing temperature for 28 days or less depending on the test age. After the initial 28-day period, half of the specimens remaining for tests at 3 months and 1 year were stored at 73 F and 100 percent relative humidity and the remaining half at 73 F and 50 percent relative humidity. Additional concretes were mixed and placed at 40 F and then, immediately after placing, the specimens in their molds were stored at 25 F. Surfaces of the specimens were covered with a double thickness of damp burlap (not in direct contact with the concrete surface) and a waterproof tarpaulin. After removal from the molds at 1 day, these specimens stored at 25 F were covered with a double thickness of initially damp burlap for 28 days or less depending upon test age. After 28 days, specimens remaining

for tests at 3 months and 1 year were treated like those stored at other temperatures.

In Part II, half of the specimens were cured for 7 days moist at the fabrication temperature while the remainder of the specimens were cured for 28 days moist at the fabrication temperature. At the end of the 7 and 28-day preliminary curing periods, each half was subdivided into two groups, one cured at 73 F and 100 percent relative humidity, and the other at 73 F and 50 percent relative humidity.

TABLE 3—AGGREGATE GRADING

Elgin sand		Elgin gravel	
Sieve No.	Percent passing	Sieve No. or size	Percent passing
4	100	1½-in.	100
8	82	¾-in.	50
16	67	⅜-in.	25
30	43	No. 4	0
50	13		
100	5	Fineness modulus of sand = 2.90	

CONCRETES

The cement content of all of the concretes was held constant at 5½ sacks per cu yd. In Part I, the net water-cement ratio for each concrete (particular cement type with or without accelerator) was such as to produce a slump of approximately 3 in. at a concrete temperature of 55 F. For concretes mixed at 40 and 73 F, the net water-cement ratio was held fixed at the value determined at 55 F, and the slump was permitted to change. In Part II, the net water-cement ratio for each concrete was such as to produce a slump of approximately 3 in. at a concrete temperature of 90 F. For the concretes mixed at 73, 105, and 120 F, the net water-cement ratio was held at the value determined at 90 F, and the slump was permitted to change. Within each part of this study, this procedure permitted a direct comparison of strengths produced at different temperatures without having to consider changes in net water-cement ratio. Continuity between Parts I and II is provided by a repetition of all of the concretes at 73 F; therefore comparisons on a percentage basis are valid despite a small difference in net water-cement ratio between the concretes in Parts I and II.

Neutralized Vinsol resin in solution was added at the mixer to entrain 4½ ± ½ percent air in all of the concretes. Changes in the amounts of air-entraining agent were necessary to maintain this constant air content when changing type of cement or concrete temperature, or when $CaCl_2$ was used.

In those mixes containing an accelerator, 2 percent flake calcium chloride (by weight of the cement) was used. The calcium chloride was dissolved in part of the mixing water prior to mixing.

CONCRETE MIXING AND FABRICATION OF SPECIMENS

All materials used in the concretes were stored at the required temperature for sufficient time prior to mixing to enable them to reach this temperature. The aggregates were weighed in the air-dried condition (moisture content

TABLE 4—CONCRETE MIX DATA

Cement content of all mixes, 5½ sacks per cu yd. Net W/C constant for each cement type. Sand percentage, all mixes, 35 percent by absolute volume of total aggregate.

Cement		Fabrication and curing temperature, deg F first 28 days	No $CaCl_2$				2 percent $CaCl_2$, by weight of cement			
Lot No.	ASTM Type		Concrete temperature after mixing, deg F	Average slump, in.	Net W/C, gal. per sack	Air content, percent	Concrete temperature after mixing, deg F	Average slump, in.	Net W/C, gal. per sack	Air content, percent
Part I, 73 F and below										
18827	I	73	75	2.4	4.82	4.5	76	2.1	4.73	4.6
		55	57	3.0	4.81	4.4	58	2.8	4.71	4.4
		40	41	3.8	4.77	4.9	43	3.8	4.70	4.7
		25*	41	4.1	4.81	4.7	43	5.1	4.71	4.6
19012	III	73	73	1.6	5.09	4.5	75	1.8	5.10	4.7
		55	59	1.7	5.08	4.8	59	2.8	5.09	5.0
		40	44	1.3	5.06	4.5	44	1.7	5.08	4.8
		25*	45	1.7	5.07	4.6	46	1.7	5.09	4.5
18830	II	73	74	2.7	4.78	4.6	75	2.0	4.47	4.7
		55	57	3.2	4.77	4.5	57	3.3	4.50	4.7
		40	41	5.3	4.75	4.8	42	5.5	4.46	4.9
		25*	40	5.4	4.77	4.3	42	5.4	4.46	4.5
Part II, 73 F and above										
18922	I	73	75	3.4	5.13	4.6	76	3.6	5.09	4.5
		90	93	3.1	5.10	4.5	95	3.1	5.08	4.7
		105	107	1.9	5.10	4.7	108	2.9	5.10	4.3
		120	121	1.3	5.11	4.4	124	1.3	5.13	4.5
18931	III	73	76	3.2	5.57	4.6	77	2.9	5.42	4.6
		90	94	3.2	5.57	4.5	96	3.2	5.42	4.5
		105	107	3.1	5.57	4.6	108	3.5	5.42	4.5
		120	121	2.2	5.55	4.6	122	2.0	5.43	4.5
18925	II	73	75	3.6	4.93	4.6	76	4.1	4.98	4.8
		90	92	3.2	4.93	4.4	94	3.1	4.98	4.6
		105	104	2.3	4.93	4.4	106	3.0	4.99	4.4
		120	121	1.2	4.93	4.5	121	1.7	4.97	4.8

*Fabricated at 40 F and placed immediately in room at 25 F.

known) and, 18 to 20 hr prior to use, inundated with a known amount of water. Excess water was drawn off and weighed immediately prior to mixing.

Each batch contained sufficient concrete to cast two 6 x 6 x 30-in. beams. The batches were mixed for 2½ min in a 1¾ cu ft capacity Lancirick mixer (horizontal open tub). Slump, air content by the pressure method, and concrete temperatures were determined immediately after mixing.

The specimens were cast in watertight steel molds, following the procedure outlined in ASTM C 192-52T. Sufficient batches were mixed (duplicates on two different days) so that two specimens were available for each test age and curing condition.

STRENGTH TESTS

Beams were tested in flexure with load applied at the third-points of an 18-in. span. Two flexural breaks were obtained for each beam. The two beam ends were tested in compression as 6-in. modified cubes. (For this particular aggregate, the ratio of 6 x 12-in. cylinder strength to 6-in. modified cube strength is 0.93.) For each test age, concrete mix, and curing procedure, two beams were tested yielding four flexural and four compressive test results for averaging.

TABLE 5—COMPRESSIVE STRENGTHS OF CONCRETE. PART I—73 F AND BELOW

Cement content of all mixes, 5½ sacks per cu yd. Net W/C constant for each cement type. Compression, 6-in. modified cubes.

Cement		Fabrication and curing temperature, deg F first 28 days	Compressive strength, psi							
							3 months†		1 year†	
Lot No.	ASTM Type		1 day	3 days	7 days	28 days	100 percent relative humidity	50 percent relative humidity	100 percent relative humidity	50 percent relative humidity
			No $CaCl_2$							
18827	I	73	1410	3120	4490	6050	6360	7070	7190	7100
		55	580	1930	3750	6250	7550	7990	8100	7720
		40	80	730	2140	5180	7120	7010	7730	6600
		25*	40	460	620	1580	6290	4300	7360	4560
19012	III	73	2640	3930	4780	5750	6320	7290	6910	7070
		55	1440	3180	4010	5470	6490	6680	6880	6740
		40	360	2000	4160	6260	7740	8090	8570	8260
		25*	140	1360	2540	4020	6430	6500	6940	6720
18830	II	73	860	1860	2760	5050	6480	7070	7440	6760
		55	400	1140	1990	4720	7190	7720	8150	7170
		40	60	560	1320	3290	7150	6360	7900	6430
		25*	40	220	640	680	6360	2780	7800	3020
			2 percent $CaCl_2$, by weight of cement							
18827	I	73	2530	3970	4960	6170	6980	7590	8340	7520
		55	1390	3210	4610	6300	7450	7840	8040	8220
		40	430	1940	3470	5350	7420	7920	8550	7540
		25*	200	950	1930	4080	6550	6530	7910	6200
19012	III	73	4440	5900	6820	7310	7820	8050	8680	8370
		55	3030	5120	5780	6740	8100	8230	8500	8280
		40	1640	3880	5320	6660	7970	7780	8040	7630
		25*	1320	3240	4400	6460	7880	7400	8210	8020
18830	II	73	1870	3300	4180	5690	7210	7640	8160	7280
		55	1030	2640	3870	5620	7470	7840	8280	7460
		40	290	1440	2740	4960	7220	7400	8460	6800
		25*	220	920	1870	3180	6920	6400	9020	5840

*Fabricated at 40 F and placed immediately in room at 25 F.
†Cured at 73 F and either 100 percent or 50 percent relative humidity after first 28 days.

All strength tests were made at a concrete temperature of 73 F. Specimens stored at temperatures other than 73 F were placed in the 73 F moist-room for temperature conditioning 1½ hr before testing. This length of conditioning period, determined on trial specimens containing thermocouples, was required specifically for those beams stored at 25 F.

DISCUSSION OF RESULTS

Tests of cements and mortars

All of the ASTM chemical and physical requirements were met by these cements (Tables 1 and 2).

Effect of temperature on slump and water requirements

Maintaining the net water per cu yd constant in both Parts I and II generally resulted in different slump at the different concrete temperatures. Fig. 1 shows an idealization of the average effect of concrete temperature on the slump of the various concretes made with the Type I and II cements. The *broken* line in this figure shows the idealized slumps as a function of concrete temperature. Approximately 1-in. decrease in slump resulted for each

20-F increase in temperature. As seen in Table 4, the slump of the concretes made with the Type III cements was not influenced by temperature. The reason for this behavior is not now apparent.

Auxiliary tests were made at some of the different temperature levels to determine the changes in water content necessary to effect a 1-in. change in slump (either increase or decrease). The *solid* line in Fig. 1 shows that as concrete temperature was increased, a greater percentage increase in water was required to effect a 1-in. change in slump than was required at lower temperatures. For example, at 73 F a 2½ percent change in water content will change the slump 1 in., while at 120 F a 4½ percent change in water content will be required.

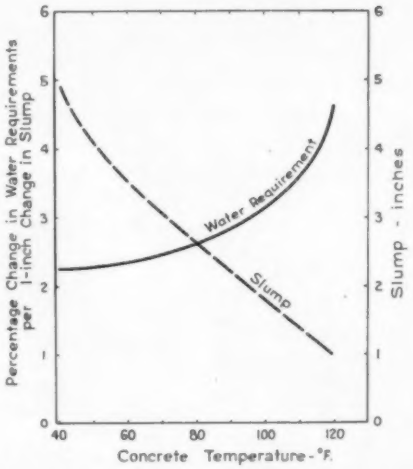

Fig. 1—Effect of concrete temperature on slump and on water required to change slump

Cement content: 5½ sacks per cu yd; 4½ ± ½ percent air; maximum size of aggregates, 1½ in.; average of data for Types I and II cements

In a concrete mix with proportions of solid ingredients fixed, changes in water content to maintain slump necessitated by changes in concrete temperature would result in concomitant changes in net water-cement ratio. These changes in net water-cement ratio would influence the strengths developed by these concretes. For example, based on the data in Fig. 1, the additional water required to maintain the 73 F slump would result in an increase in net water-cement ratio of about 0.6 gal. per sack for the 120-F concretes. In order to maintain the original water-cement ratio, and thereby not sacrifice strength, an increase of about 0.6 sacks of cement per cu yd would be required for the 120-F concretes. This point will be discussed further later in this report.

Effect of initial and curing temperature on strength development

In order to facilitate the presentation and discussion of the large amount of strength data (Tables 5, 6, 7, 8) obtained, Fig. 2 and 3 have been prepared using the strengths expressed as percentages of the 73-F strengths. Fig. 2 shows the effect of temperature on compressive strength, Fig. 3 the effect on flexural strength. These data are for concretes cured moist the first 28 days. In Part II, a 7-day moist-curing period at the fabrication temperature was included for comparison with the 28-day moist-curing period at the fabrication temperature. The qualitative interpretation of the results for this 7-day group is similar to that for the 28-day group.

TABLE 6—COMPRESSIVE STRENGTHS OF CONCRETES MADE AT DIFFERENT TEMPERATURES. PART II—73 F AND ABOVE

Cement content of all mixes, 5½ sacks per cu yd. Net W/C constant for each cement type. Specimens, 6-in. modified cubes.

Cement		Fabrication and curing temperature, deg F, first 7 or 28 days	Compressive strength, psi													
			Moist at fabrication temperature				28 days moist at fabrication temperature, remainder at 73 F				7 days moist at fabrication temperature, remainder at 73 F					
							3 months		1 year		28 days		3 months		1 year	
							Relative humidity, percent				Relative humidity, percent					
Lot No.	ASTM Type		1 day	3 days	7 days	28 days	100	50	100	50	100	50	100	50	100	50
No $CaCl_2$																
18922	I	73	1150	2600	4060	5440	6000	6840	6450	7230	5440	5700	6000	6190	6450	6270
		90	1490	3090	4160	5240	5700	6480	6240	6180	4820	5380	5460	5710	6320	5640
		105	1850	3200	3850	4720	5250	5820	5860	6160	4200	4700	4920	5600	5820	5690
		120	2040	2840	3440	4110	4730	5510	5320	5300	4080	4340	4320	5020	5410	5030
18931	III	73	2210	3460	4610	5460	5970	6430	6440	6800	5460	6280	5970	6120	6440	6580
		90	2760	4290	5010	5710	5910	6950	6860	7040	5610	6030	5940	6500	6920	6220
		105	3010	3900	4580	5120	5660	6750	6190	6660	5210	5660	5300	5980	6060	6110
		120	3280	3850	4160	4720	5040	6140	5420	5880	4490	5040	5000	6010	5720	5760
18925	II	73	990	2180	3680	5230	6570	7070	7620	6940	5230	5840	6570	6460	7620	6160
		90	1250	2720	3900	5780	6400	7150	6610	6890	5190	5470	6090	5880	6400	5780
		105	1500	3070	4420	5330	6120	6740	6630	7020	4990	5570	5760	5850	6490	5880
		120	1950	3060	3930	4820	5760	6460	5780	6190	4720	5040	5300	5690	5630	5620
2 percent $CaCl_2$																
18922	I	73	2300	3490	4420	5420	6460	7420	7730	7360	5420	6620	6460	6940	7730	6940
		90	2540	4010	4810	5720	6350	6980	7010	7000	5400	5960	6350	6740	7000	6640
		105	2800	3660	4630	5410	6020	6760	6730	7050	5030	5790	5400	5900	6660	6280
		120	3060	3720	4320	5130	5720	7090	6320	6510	4930	5420	5440	6230	6280	5990
18931	III	73	3890	4940	5870	6450	7200	8040	7680	7700	6450	7410	7200	7640	7680	7040
		90	4490	5380	6070	6790	6500	7190	6880	7580	6800	7620	7300	6910	7690	7030
		105	4410	5200	5940	6170	6940	7970	6930	7680	6070	6750	6200	7020	7000	7100
		120	4410	4940	5600	6010	6780	8060	6730	7540	5970	6850	6240	7120	6920	7040
18925	II	73	1960	3300	4280	5590	6680	7400	7700	7200	5590	6190	6680	6630	7760	6220
		90	2350	4110	4820	6120	6970	7430	7730	7610	5540	6020	6490	6850	7510	6580
		105	2510	3630	4610	5620	6400	7360	6940	7290	5200	5830	5760	5750	6930	6280
		120	2580	3660	4820	5740	6320	6660	6580	7440	4780	5850	5850	6830	6720	6140

Fig. 2 shows all of the compressive strength data for the three types of cement with and without accelerator expressed as percentages of the strengths developed at 73 F for each test age. This figure shows the accelerating effect of temperatures above 73 F on the early-age strengths with a sacrifice however in strengths at the later ages. On the other hand, the concretes placed and cured at temperatures below 73 F, while showing lower strengths at the

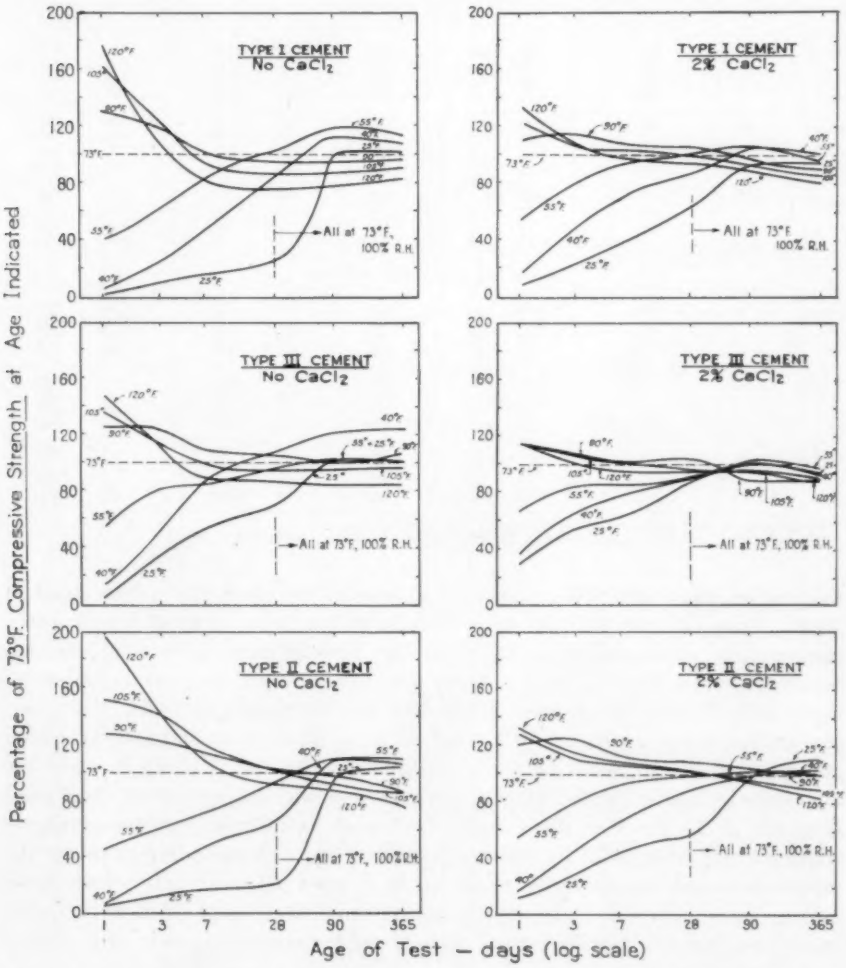

Fig. 2—Effect of temperature on compressive strength of concretes made with different types of cement with and without an accelerator

Air content of all concretes, 4½ ± ½ percent (neutralized Vinsol resin solution added at mixer); cement content of all concretes, 5½ sacks per cu yd. Within the individual boxes, the net water-cement ratios of the concretes are approximately equal (see tabulated data on mix characteristics)

TABLE 7—FLEXURAL STRENGTHS OF CONCRETE. PART I—73 F AND BELOW

Cement content of all mixes, 5½ sacks per cu yd. Net W/C constant for each cement type. Flexure, 1/3-point loading on 18-in. span.

Cement		Fabrication and curing temperature, deg F first 28 days	Flexural strength, psi							
							3 months†		1 year†	
Lot No.	ASTM Type		1 day	3 days	7 days	28 days	100 percent relative humidity	50 percent relative humidity	100 percent relative humidity	50 percent relative humidity
			No $CaCl_2$							
18827	I	73	250	460	555	665	740	685	740	810
		55	110	320	495	640	790	790	850	790
		40	10	135	335	580	710	705	845	660
		25*	10	95	125	265	720	490	790	510
19012	III	73	360	510	535	660	710	725	670	670
		55	230	435	515	600	735	730	715	640
		40	60	290	530	650	735	815	770	760
		25*	20	190	340	415	665	635	725	535
18830	II	73	160	315	445	645	735	730	820	790
		55	75	205	335	530	795	780	860	750
		40	10	100	245	460	760	725	790	700
		25*	10	50	135	165	705	400	820	335
			2 percent $CaCl_2$ by weight of cement							
18827	I	73	340	500	540	630	750	705	795	800
		55	215	385	480	570	745	770	820	805
		40	80	265	435	550	750	745	760	735
		25*	35	170	270	425	690	660	780	710
19012	III	73	435	580	625	690	720	800	745	705
		55	390	520	560	645	720	755	755	655
		40	240	440	570	585	715	720	745	630
		25*	220	390	490	520	715	690	770	645
18830	II	73	305	400	460	585	730	750	800	795
		55	180	330	425	565	770	780	800	750
		40	60	240	340	475	720	700	840	685
		25*	45	160	265	350	710	630	880	590

*Fabricated at 40 F and placed immediately in room at 25 F.
†Cured at 73 F and either 100 percent or 50 percent relative humidity after first 28 days.

early ages, show strengths at the later ages in excess of those developed at 73 F. This was true even for the concretes mixed and cast at 40 F and stored immediately after casting at 25 F for the first 28 days. For example, the concrete made with Type I cement (no accelerator) developed during 28 days at 25 F only 26 percent of the strength developed at 73 F. However, subsequent moist curing at 73 F resulted in a compressive strength at 1 year 102 percent of that resulting from continuous moist curing at 73 F, while the concretes at 120 F for the first 28 days showed only 82 percent of the 1-year strength at 73 F. For those concretes cured initially at low temperatures followed by curing at 73 F, 1-year strengths close to or exceeding those for the concretes cured continuously moist at 73 F were attained only when moist curing was employed. Air drying during this subsequent 73-F period generally resulted in lower strengths, particularly for concretes made with Type I and II cements.

The concretes made with the three different types of cement reacted to temperature in a similar manner, but differed somewhat in degree of reaction particularly at the earlier ages. For example, the 120-F concrete made with Type II cement had a 1-day compressive strength 197 percent of that at 73 F,

TABLE 8—FLEXURAL STRENGTHS OF CONCRETES MADE AT DIFFERENT TEMPERATURES. PART II—73 F AND ABOVE

Cement content of all mixes, 5½ saks per cu yd. Net W/C constant for each cement type.
Specimens, 6 x 6 x 30-in. beams. Third-point loading on an 18-in. span.

Cement		Fabrication and curing temperature, deg F, first 7 or 28 days	Flexural strength, psi													
			Moist at fabrication temperature				28 days moist at fabrication temperature, remainder at 73 F				7 days moist at fabrication temperature, remainder at 73 F					
							3 months		1 year		28 days		3 months		1 year	
							Relative humidity, percent				Relative humidity, percent					
Lot No.	ASTM Type		1 day	3 days	7 days	28 days	100	50	100	50	100	50	100	50	100	50
No $CaCl_2$																
18922	I	73	220	405	530	675	780	725	790	830	675	585	780	745	790	725
		90	265	420	520	585	720	640	675	735	610	525	705	670	690	685
		105	285	420	505	610	705	540	710	760	600	505	640	625	690	660
		120	295	400	450	545	585	485	645	665	550	410	570	530	615	590
18931	III	73	340	450	515	600	725	680	700	740	600	595	725	715	700	700
		90	400	525	545	615	725	695	720	815	660	540	660	710	710	670
		105	400	455	465	565	630	560	690	680	575	495	610	625	640	630
		120	385	430	485	510	605	445	605	600	575	465	610	550	635	605
18925	II	73	190	360	490	635	740	725	855	735	635	580	740	755	855	695
		90	215	390	540	600	735	670	790	750	645	505	740	675	770	645
		105	265	410	515	660	730	640	780	770	650	510	720	635	745	660
		120	320	390	480	560	640	500	690	720	605	470	610	525	660	630
2 percent $CaCl_2$																
18922	I	73	295	440	515	605	750	710	760	805	605	570	750	700	760	740
		90	335	435	525	620	725	675	730	780	640	560	700	700	750	680
		105	345	390	460	605	665	585	695	685	585	495	630	600	645	645
		120	375	395	440	545	625	625	680	700	560	500	610	600	635	655
18931	III	73	410	490	560	640	705	720	725	715	640	630	705	695	725	665
		90	410	485	560	625	740	790	720	740	630	630	690	730	750	655
		105	420	465	480	540	630	685	690	735	600	620	645	665	660	665
		120	450	430	460	580	665	645	675	765	630	580	650	665	675	695
18925	II	73	290	430	475	590	705	720	715	740	590	595	705	680	715	665
		90	340	440	495	665	740	725	800	780	625	575	670	730	765	705
		105	350	400	420	605	700	690	750	740	595	555	665	660	730	665
		120	320	395	440	590	680	655	695	760	620	540	655	655	710	660

while the 120-F concrete made with the Type III cement had a 1-day compressive strength 148 percent of that at 73 F. At 1 year, however, the strengths of these same concretes were 77 percent and 84 percent, respectively, of the 73-F strengths.

The concretes made with and without the accelerator reacted similarly to temperature; however, the effect of temperatures above 73 F on the early

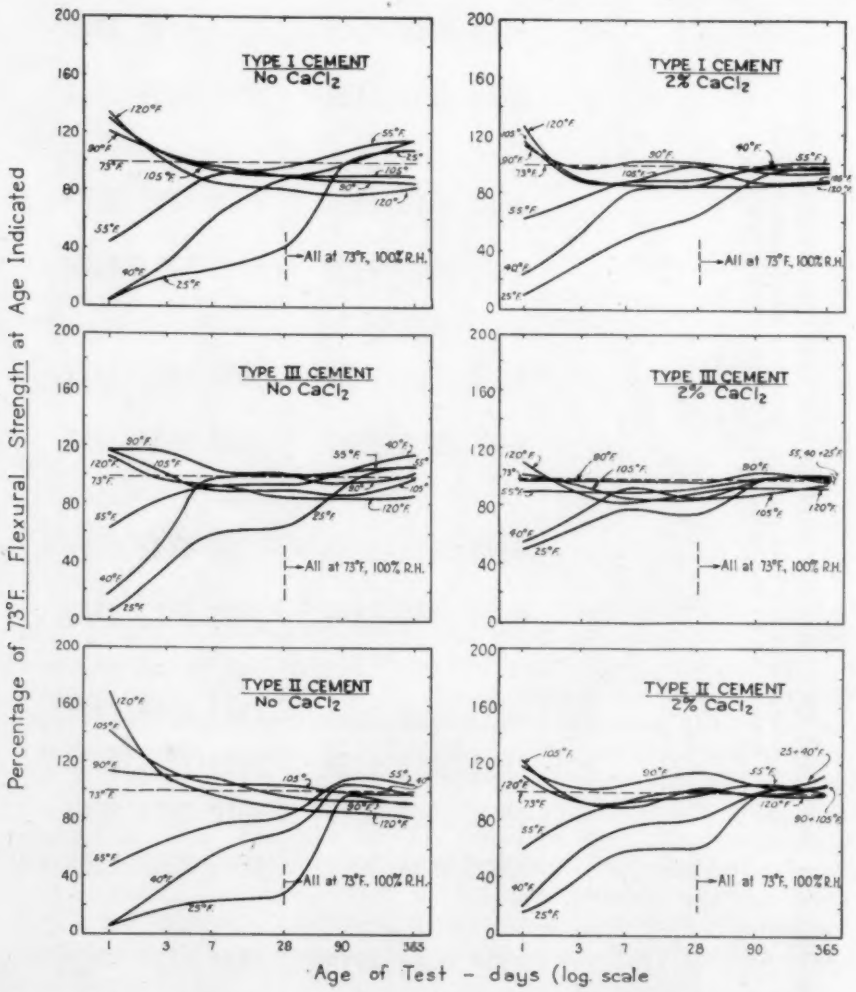

Fig. 3—Effect of temperature on flexural strength of concretes made with different types of cement with and without an accelerator

Air content of all concretes, 4½ ± ½ percent (neutralized Vinsol resin solution added at mixer); cement content of all concretes, 5½ sacks per cu yd. Within the individual boxes, the net water-cement ratios of the concretes are approximately equal (see tabulated data on mix characteristics)

and ultimate strengths was somewhat moderated by the use of calcium chloride.

The data indicate that there is a casting and curing temperature which may be considered optimum with regard to the ultimate strength developed. For the Type I and II cements without accelerator this optimum is 55 F. For the Type III cement without accelerator, the optimum is 40 F. With calcium chloride as an accelerator, these optimum temperatures appear somewhat lower.

Temperature seems to affect the hydration and hence development of strength in two ways. First, there is the known effect of temperature on any chemical reaction, the rate of reaction increasing as temperature increases. Since in a general sense the strength of like concrete mixes is proportional to the amount of hydration, this greater rate of reaction at the higher temperatures accounts for the higher strength at early ages. However, a second factor may be that the type of hydration product obtained or the physical make-up of the product is influenced by the temperature during this hydration. Lower temperatures may be conducive to a better hydration product or better physical structure of the product. The role of gypsum as a function of temperature needs further investigation in this regard.

Fig. 3 shows the flexural strength data for these same concretes. The preceding discussion of compressive strength data applies equally to the flexural strength data. Flexural strengths at early ages increased with increase in temperature. At later ages, the effect of temperature was reversed. Those concretes made and cured at the lower temperatures showed highest flexural strengths at 1 year. The optimum temperatures for flexural strength development appear to be the same as those for compressive strength.

Effect of age and temperature on strength

Many investigators have proposed that differences in curing temperature can be accounted for in interpreting strength development data by the use of a parameter which includes both age and temperature.* This has led to the use of the term "degree-days." The number of "degree-days" representing a particular amount of curing is obtained by multiplying the curing time in days by the temperature, in deg F, *above* the minimum temperature for hydration. The choice of this minimum temperature for hydration is somewhat arbitrary, however, since it is influenced by the characteristics of the cement used, initial concrete temperature, mix proportions, and amount of protection afforded by forms. Some exploratory tests indicated that the concretes in this study mixed and placed at 40 F and then immediately stored at 10 F were capable of developing a small amount of strength during 28 days at this temperature. On the basis of these tests, this temperature of 10 F was selected as the minimum temperature for hydration for these concretes.

*Nurse, R. W., "Steam Curing of Concrete," *Magazine of Concrete Research* (London), V. 1, No. 2, June 1949, pp. 79-88.

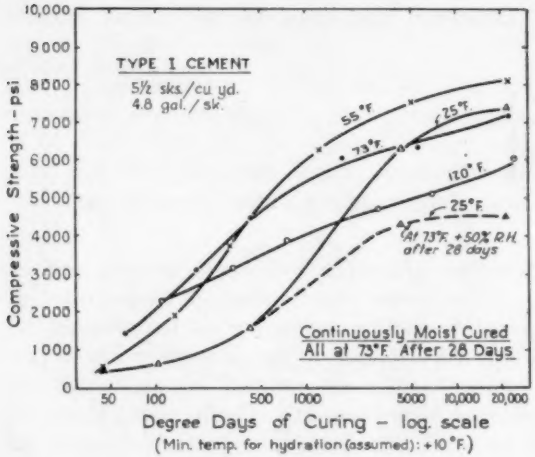

Fig. 4—Relationship between degree-days of curing and compressive strength

Air content of concretes, 4½ ± ½ percent. No $CaCl_2$. Strengths of 120-F concretes corrected for difference in water-cement ratio between these concretes and the 55 and 73-F concretes

Fig. 4 shows the relationships between degree-days of curing and the compressive strength of the 25, 55, 73, and 120-F concretes made with Type I cement. The separate relationships for each initial concrete temperature indicate poor correlation of strength with this parameter. The fact that the 55-F concretes produced higher strengths at the later ages than the 73-F concretes and still higher strengths than the 120 F concretes precludes significant correlation on a basis as simple as degree-days.

The data for the Types II and III cements and for flexural strengths are similar to those shown in Fig. 4. For the concretes containing calcium chloride as an accelerator, somewhat better correlation of strength with degree-days was obtained.

A factor not considered in this type of correlation is the presence or absence of water during curing. Data for the concretes at 25 F for the first 28 days indicate clearly that strength developed during subsequent curing at 73 F depends considerably upon the presence of water during this period. This was more marked for these 25 F concretes than for the concretes made and cured at 40 F and higher. Tables 5, 6, 7, and 8 show strength data for the 50 percent relative humidity storage condition along with the data for the continuously moist-cured concretes.

Effect of addition of water to maintain slump

In discussing the effect of temperature on slump and water requirements, it was pointed out that the addition of water to concrete at 120 F to obtain the same slump as at 73 F would increase the net water-cement ratio of the mix, unless additional cement were added to offset this increase in water. Fig. 5 shows the compressive strength developed by the 55, 73, and 120-F concretes at the same net water-cement ratio. The strengths of the 120-F concretes were corrected for the slight difference in water-cement ratio between these concretes and the 55 and 73-F concretes, this procedure being

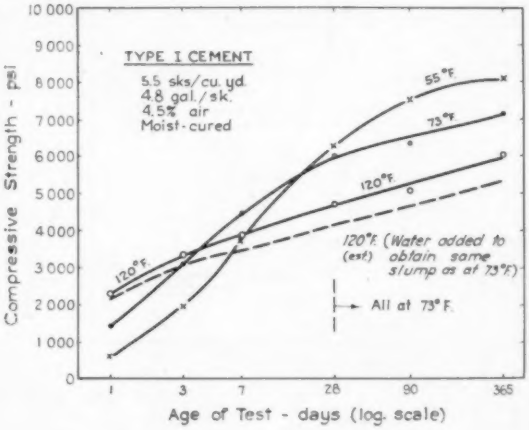

Fig. 5—Effect of curing temperature and addition of water to maintain slump on the compressive strength

Strengths of 120-F concretes corrected for difference in water-cement ratio between these concretes and the 55 and 73-F concretes. Dashed line illustrates effect of addition of water to obtain same slump as at 73 F, without addition of cement required to maintain same net water-cement ratio

possible since the 73-F concretes were prepared in Parts I and II. The effect on strength of adding water to the 120-F concretes in order to obtain the same slump as the 73-F concretes is shown by the dashed line in this figure. The strengths shown in the dashed line were estimated from the change in net water-cement ratio which would result from this procedure, the change being based on the water-requirement data in Fig. 1. At 28 days, the effect of this added water is to reduce strength 500 psi below that of the original 120-F concrete. While this is approximately a 10 percent reduction, it is worth noting that the original 120-F concrete shows 1250 psi less strength at 28 days than the 73-F concrete at the same net water-cement ratio. Occurrence of low strengths during hot weather may therefore be attributed to a large extent to two factors: increasing water without a concomitant increase in cement, and the effect of initial temperature on strength at the later ages.

Effect of cement type on strength development

The effect of type of cement on compressive and flexural strength development at the various test ages at the different temperatures can be determined from data shown in Tables 5, 6, 7, and 8.

Concretes made with Type III cements showed higher strengths at early ages than concretes made with Type I cements, while concretes made with Type II cements showed lower strengths at early ages than concretes made with Type I cements. These strength differences tended to disappear at the later ages, however.

These data afford a means of comparing the strength development of the concretes made with Type II and Type III cements with those made with Type I cements. For example, at 40 F for concretes made without calcium chloride the 7-day compressive strength of the concrete made with Type III cement (4160 psi) is 195 percent of that made with Type I cement (2140 psi), while for the Type II cement (1320 psi) 62 percent of the Type I strength is developed. For flexural strengths, the respective percentages are 158 percent

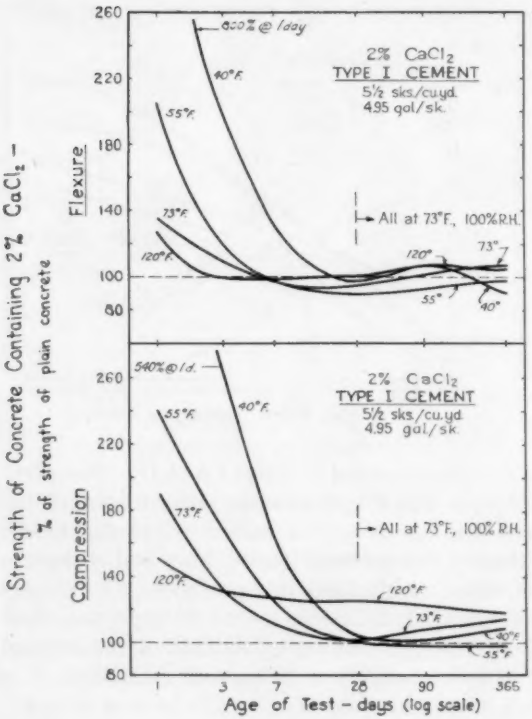

Fig. 6—Effect of calcium chloride on strength development of concretes made with Type I cement

Air content, 4½ ± ½ percent (neutralized Vinsol resin solution added at mixer); average net water-cement ratio 4.95 gal. per sack

and 73 percent. These data provide typical information on strength development as affected by cement type over a rather wide temperature range.

Effect of calcium chloride on strength development

Information on the effect of calcium chloride on flexural and compressive strengths of these concretes can be determined from data in Tables 5, 6, 7, and 8.

Fig. 6 shows the effect of calcium chloride on flexural and compressive strengths of the 40, 55, 73, and 120-F concretes made with the Type I cements. Each solid line is a comparison by itself with the dashed 100-percent line, which represents the strength attained without calcium chloride. For example, at 1 day for the 55-F concretes, the compressive strength of the concrete made with 2 percent calcium chloride was 240 percent of that of the mix without calcium chloride. For the 120-F concretes, the use of calcium chloride produced a compressive strength 150 percent of that of the mix without calcium chloride. This means that at 55 F, the calcium chloride produces a proportionately greater increase in strength than it does at 120 F.

For both flexural and compressive strengths, the increases effected by use of calcium chloride decreased with an increase in the age of concrete and became quite small at 28 days and later. For the 120-F concrete, the use of

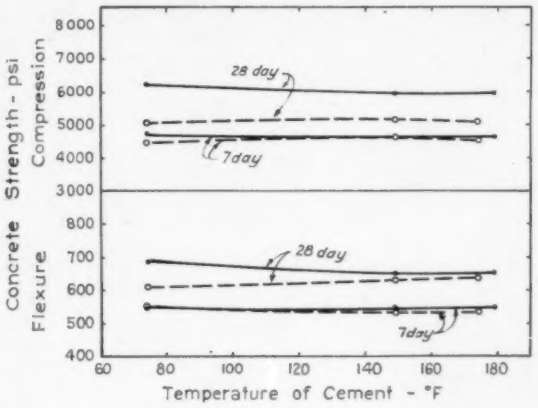

Fig. 7—Effect of temperature of cement on concrete strength

Solid line represents concrete temperature 76 F, cured at 73 F, 100 percent relative humidity. Dashed line, concrete temperature 104 F, cured 7 days at 105 F and 100 percent relative humidity, remainder at 73 F and 100 percent relative humidity

Type I cement, Lot 18922; 5½ sacks per cu yd, 4.9 gal. per sack
Specimens: 6 x 6 x 30-in. beams
Flexure: Third-point loading, 18-in. span
Compression: 6-in. modified cubes

calcium chloride produced a smaller increase in compressive strength at the early ages; however, this increase appears to persist longer than at the other temperatures. Despite the persistence of the increases effected at 120 F, these 120-F concretes showed lower strengths at 28 days and later than the 55 or 73-F concretes made with calcium chloride, as can be seen in Fig. 2 and Fig. 3. This was true for the three types of cement and for both flexural and compressive strength.

At the later ages, concretes made with calcium chloride showed compressive strengths equal to or somewhat greater than those without the accelerator. The later-age flexural strengths, however, were reduced somewhat when calcium chloride was used. The reductions encountered at 28 days and later ages ranged from negligible values to about 10 percent, these reductions showing no particular relationship with making and curing temperatures. The beneficial effects of the accelerator for the low temperature early-age concretes are relatively more important than these generally small decreases in flexural strength at the later ages.

Effect of temperature of cement on strength

High cement temperature has sometimes been blamed for a lowering of concrete strength, in spite of evidence to the contrary.*

Strength tests were made of concretes prepared with the Type I cement (Lot No. 18922) at three different cement temperatures: 74, 148, and 177 F. Cements at each of these temperatures were used in preparing concretes which had temperatures at the end of the mixing period of 76 F and 104 F. These final concrete temperatures were attained by adjusting mixing water temperatures, using the aggregates maintained at the same temperature as that of the concrete after mixing. The 76-F concretes were cured continuously moist at 73-F. The 104-F concretes were cured continuously moist for 7 days at 105 F followed by 21 days moist at 73 F.

*Report of Working Committee of ASTM Committee C-1, *Proceedings*, ASTM, V. 32, Part I, 1932, p. 298.

TABLE 9—AIR-ENTRAINING AGENT REQUIREMENTS

Cement type	$CaCl_2$, percent by weight	Amount of air-entraining agent, ml per lb cement					
		40 F	55 F	73 F	90 F	105 F	120 F
I	0	1.5	1.6	2.2	2.8	3.0	3.6
	2	1.0	1.2	1.8	2.6	2.9	5.0
III	0	3.6	4.4	4.7	6.4	7.0	9.8
	2	2.8	2.8	3.3	4.9	6.2	11.0
II	0	1.7	1.7	2.4	3.0	3.3	4.5
	2	1.0	1.3	1.7	2.5	3.2	5.2

Fig. 7 shows the flexural and compressive strengths of these concretes at 7 and 28 days as a function of cement temperature at the time of mixing. All of these concretes had the same cement content and the same net water-cement ratio, therefore the strengths are directly comparable. The average slump of the 74-F concretes was approximately 3 in., while for the 104-F concretes the average slump was slightly less than 1 in. It is apparent from Fig. 7 that the cement temperature in itself exerts no significant influence on the strength of the concrete so long as the comparison is made on the basis of equal concrete temperature. The temperature of the concrete during curing has by far more influence than the temperature of the cement, as shown by the 28-day strengths. The concretes made at 104 F and cured 7 days at 105 F and 21 days at 73 F had compressive strengths about 1000 psi lower than the concretes made and cured at normal temperatures. The same conclusions can be drawn from the flexural strength data.

These results show that cement temperature was of concern only insofar as it affected the temperature of the concrete after mixing. However, cement temperature plays only a small role in the final concrete temperature because of the relatively small amount used and its low specific heat. The temperature of the aggregates, by virtue of their large quantity, and the temperature of the water with its high specific heat exert more influence on the final concrete temperature than the cement.*

Effect of concrete temperature on Vinsol resin requirements

The air contents of all of the concretes in this study were maintained within the range of $4\frac{1}{2} \pm \frac{1}{2}$ percent. Neutralized Vinsol resin solution was added at the mixer as the air-entraining agent.

Changes in concrete temperature required changes in the amount of air-entraining agent per unit of cement needed to entrain $4\frac{1}{2} \pm \frac{1}{2}$ percent air. The amount of agent required increased with an increase in concrete temperature. This increased demand with temperature was accentuated somewhat in these tests since the water content per cu yd was held constant; consequently slump decreased with increased temperature. The amount of air entrained per unit of agent generally increases with an increase in concrete slump up to about 5 or 6 in. The air-entraining agent requirements shown in Table 9 for the different concrete temperatures are therefore influenced by slump in addition to temperature.

*Swayze, M. A., "Hot Cement Effect on Concrete Mix Temperature," ACI JOURNAL, Dec. 1947, *Proc.* V. 44, p. 330.

At all temperatures except 120 F, the concretes made with calcium chloride as the accelerator required slightly less air-entraining agent than those without calcium chloride. At 120 F the reverse was true. The requirements for the Type I and II cements were approximately equal, while the Type III cements required greater quantities of agent than either the Type I or II.

SUMMARY AND CONCLUSIONS

Based on these laboratory tests of air-entrained concretes mixed, placed, and cured at different temperatures, the following statements appear valid:

1. Concrete slump generally decreased with an increase in concrete temperature when the water content was held constant. Increasing the water content of concrete without increasing the cement content is one of the reasons for reductions in strength during hot weather.
2. At 1, 3, and 7 days, concrete strengths increase with an increase in the initial and curing temperature of the concrete.
3. Increasing the initial and curing temperatures results in considerably lower strengths at 3 months and 1 year.
4. Temperature influences flexural strength development in much the same manner as it does compressive strength development.
5. Strengths of concretes made with the three cement types used in this study were influenced in a like manner by temperature; differences were in degree only.
6. These tests indicate that there is a temperature during the early life of the concrete which may be considered optimum with regard to strength at later ages, or more strictly, at comparable degrees of hydration. This temperature is influenced somewhat by cement type. For Types I and II this temperature is 55 F, for Type III it is 40 F.
7. Concrete strengths did not correlate well with the simple index known as "degree-days."
8. For concretes with calcium chloride added, strength increases due to the accelerator are proportionately greater at early ages and lower temperatures. The use of calcium chloride frequently resulted in flexural strengths at later ages somewhat lower than for comparable concretes without calcium chloride. Maximum reductions noted were of the order of 10 percent.
9. In these tests, cement temperature by itself exerted no significant influence on the strength of the concrete except insofar as it affected the final temperature of the concrete after mixing. It is concrete temperature and curing temperature which influence strength development and not cement temperature.
10. To maintain a constant amount of entrained air, greater amounts of air-entraining agent per unit of cement are required as concrete temperature is increased and slump decreased.

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Title No. 54-63

Precast Reinforced Concrete Slab Bridges with Stiffened Edges*

By ANDREW GALLIA†

SYNOPSIS

A method is given for the design of a simply supported highway bridge constructed of solid precast reinforced concrete slab units and two curb units. It is shown that this design improves the transverse wheel load distribution and an economical and practical bridge is achieved.

The paper is divided into (a) a discussion of the variables affecting the transverse distribution of wheel loads, such as transverse stiffness factor, edge stiffness factor, and effective stiffness factor; (b) a design procedure investigating the effects of the longitudinal and transverse live load moments in the slab; and (c) numerical examples showing economy of the proposed structure.

INTRODUCTION

Practical experience shows that it is uneconomical to place precast slab units side by side to form a slab bridge. The wheel load distribution is limited to the width of the precast element, which has to be designed as a freely supported beam.

This paper presents a satisfactory solution for the design and construction of a precast slab bridge on the basis of an analysis of all the factors involved.

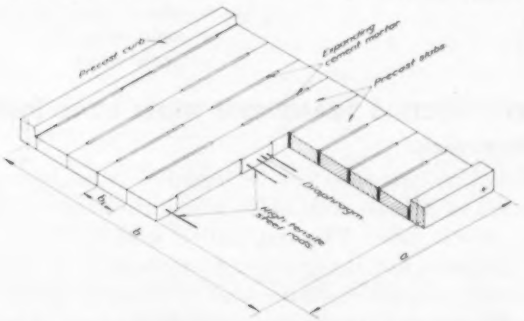

Fig. 1—Precast reinforced concrete slab bridge with stiffened edges

This design requires no *in situ* concrete but uses several precast interior slab units and two exterior curb units (Fig. 1). Since only one group of wheel loads, the front axles of the H20-S16-44 trucks, are considered on the bridge deck, the bridge span is limited to 24 ft.

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†Member American Concrete Institute, Public Works Department, Hobart, Tasmania, Australia.

When all precast units are in position, high tensile steel rods and nuts are used at midspan and over supports to tie the entire structure together. Each nut may be tightened up with a manual or pneumatic impact wrench depending on the size of the tie rod.

Expanding cement mortar is required between adjacent slab units, which will exercise a lateral thrust on the slabs and serve as a shear connection.

Notation

a = span of bridge
b = distance from center to center of curbs
b_1 = spacing of precast units or width of them
E = modulus of elasticity of concrete
I_x = moment of inertia of longitudinal interior member
I_x' = moment of inertia of longitudinal exterior member
I_y = moment of inertia of transverse member
D_x = flexural stiffness in longitudinal direction = EI_x
D_y = flexural stiffness in transverse direction = EI_y
m = number of transverse diaphragms
i = coefficient = $0.493\ (m + 1)$
X = transverse stiffness factor

$$= \frac{I_y}{I_x}\left(\frac{a}{2b_1}\right)^3 i$$

H = edge stiffness factor = $\dfrac{I_x'}{I_x}$
N = effective stiffness factor = $\sqrt{D_x D_y}$
P = wheel load
M = bending moment due to dead and live load
M_F = bending moment due to eccentricity of prestress, F
Q = torque
F_o = initial prestress force
F = effective prestress force
h = depth of section
d = effective depth of section
A_c = area of concrete section (steel area not deducted)
A_s = steel area
Z = section modulus
R = constant used in reinforced concrete design
p = ratio of tensile reinforcement
f_c = compressive stress
f_s = tensile stress

VARIABLES AFFECTING TRANSVERSE WHEEL LOAD DISTRIBUTION

Transverse stiffness factor

A grid supported on four sides can be regarded as the basic structure for a bridge constructed of precast slab units and curb units tied together transversely by high tensile rods. The longitudinal grid members are represented by the precast slab units, and the transverse members by "hidden" diaphragms. It will be sufficiently accurate to assume that the flexural rigidity of a transverse member, $D_y = EI_y$, is equal to the flexural rigidity of an interior longitudinal member, $D_x = EI_x$. Assuming $I_x = I_y$, the equation for the transverse stiffness factor, X,[5] can then be written as

$$X = \frac{I_y}{I_x}\left(\frac{a}{2b_1}\right)^3 i = \left(\frac{a}{2b_1}\right)^3 i \text{ (see Reference 5)}$$

where $i = 0.493\ (m + 1)$ [see Reference 3].*

*The coefficient i as given by Leonhardt and Andrä[1] is 1.0 for one to two diaphragms, 1.6 for two to four diaphragms, and 2.0 for five to six diaphragms. On the other hand, Massonet[2] suggests the equation used in this report, $i = 0.493\ (m + 1)$, resulting in $i = 0.986$ for one diaphragm, 1.479 for two, 1.972 for three, 2.464 for four, and 5.423 for ten diaphragms.

Therefore, for a structure with ten diaphragms ($m = 10$), we have

$$X = 0.68\,(a/b_1)^3$$

for $m = 3$

$$X = 0.25\,(a/b_1)^3$$

for $m = 1$

$$X = 0.12\,(a/b_1)^3$$

The effect of the transverse stiffness X on the bending moment M_x is shown in Fig. 2. A bridge structure with no transverse connection is indicated by the curve $m = 0$; a structure with one or three diaphragms by $m = 1$ or $m = 3$, respectively. The curve $m = \infty$, $H = \infty$ on the other hand represents a monolithic rectangular slab supported on four edges.

Edge stiffness factor [$H = I_x'/I_x$][5]

In a bridge constructed of precast slab units of equal stiffness tied together in the transverse direction, the exterior slab units can be more heavily loaded than the interior ones. Since the design is governed by these more heavily loaded exterior units, it leads to over-designed interior units and results in uneconomical bridge structures (Curve $m = 1$, $H = 1$ in Fig. 2). By using two precast curb units with greater stiffness than the slab units (Curve $m = 1$, H = 2 in Fig. 2), both curb units and slab units can be designed to attain their full working stress, resulting in much more economical use of material.[5]

Effective stiffness factor

Table 1 shows how the stiffness factor,[2] $N = \sqrt{D_x\,D_y}$, varies with the type of bridge structure. The two extreme cases are the monolithic slab supported on four sides with $N = Eh^3/[12(1 - \mu^2)]$ and a structure built of simply supported slab units, but not interconnected, with $N = 0$. The proposed structure lies between these two cases.

To achieve the greatest possible effective stiffness, expanding cement mortar may be used in the joints between precast units along with the transverse high tensile steel tie rods.

DESIGN PROCEDURE

Live load moments are computed as the effects of four equal wheel loads. For computing the moments from which the curves of this paper were drawn, the flexural rigidity of the hidden diaphragm was assumed to be equal to the flexural rigidity of a longitudinal slab unit. The width b_1 of the slab units was taken as 3 ft, but the analysis is applicable with slight modification to precast units of $b_1 < 3$ ft.

The effectiveness of a diaphragm is greatest in the center of the bridge span and for structures with limited span (24 ft).[5] Only one central diaphragm was considered here. Tie rods were employed over each support for practical reasons only.

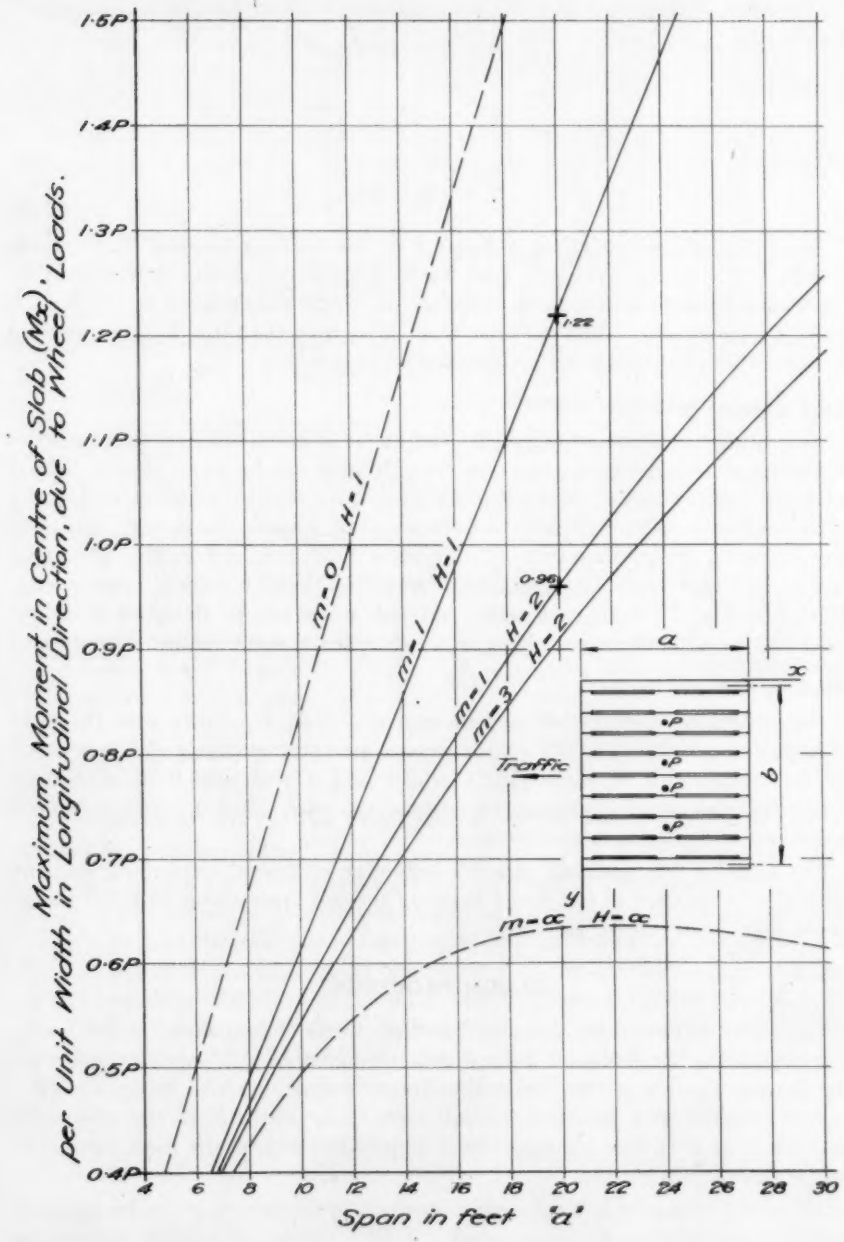
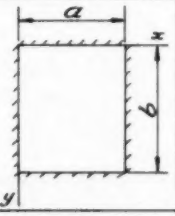
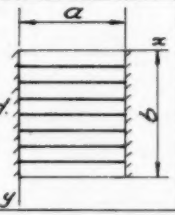
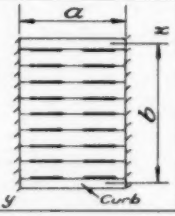


Fig. 2—Variation of live load moment at center of slab with span of slab for 24-ft roadway

TABLE 1—VARIATION OF EFFECTIVE STIFFNESS FACTOR WITH TYPE OF SLAB BRIDGE

Type of Structure	Flexural Stiffness D_x	Flexural Stiffness D_y	Effective Stiffness $N=\sqrt{D_x \cdot D_y}$
Slab supported on four sides. 	$\frac{Eh^3}{12(1-\mu^2)}$	$\frac{Eh^3}{12(1-\mu^2)}$	$\frac{Eh^3}{12(1-\mu^2)}$
Slab Units simply supported. 	$\frac{E\sum I_x}{b}$	0	0
Interconnected Slab and Curb Units. 	$\frac{E\sum I_x}{b}$	$\frac{E\sum I_y}{a}$	$0 < N < \frac{Eh^3}{12(1-\mu^2)}$

Bending moments, M_x, in precast slab units in longitudinal direction

The curves in Fig. 2 give the bending moments in the longitudinal direction per unit width. The curves $m = 1$, $H = 2$ and $m = 3$, $H = 2$ were computed for concentric loading, i.e., the center line of loading on center line of bridge. A check for cases when wheels are shifted toward a curb will be satisfactory in all practical cases. Curve $m = 1$, $H = 1$ was computed for eccentric loading, i.e., an outside wheel near the edge of the slab.[4] Moment coefficients are given in terms of P.

Bending moments, M_y, in transverse central diaphragm

In Fig. 3, the curve $m = 1$, $H = 2$ represents a structure with one diaphragm in which the ratio of the flexural rigidity of the curb unit to the slab unit is 2. The curve $m = 1$, $H = 1$ represents a structure with one diaphragm but having curb and slab units of equal stiffness. The moment is given by the coefficient of Pb_1.

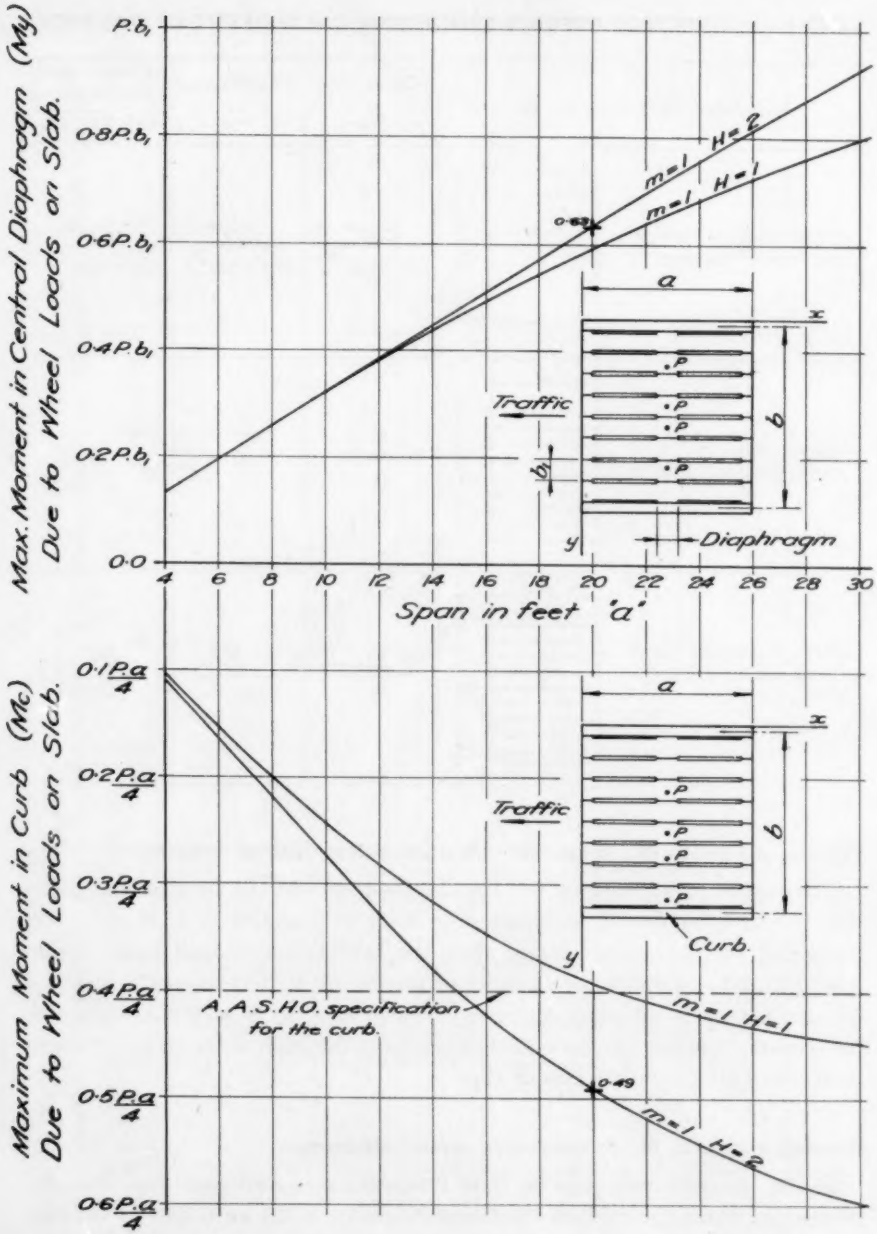


Fig. 3 (top)—Live load moment in central diaphragm with span of slab for 24-ft roadway

Fig. 4 (bottom)—Live load moment in curb with span of slab for 24-ft roadway

TABLE 2—PROPERTIES OF STRUCTURAL MEMBERS IN EXAMPLES

Property	Slab unit	Diaphragm	Curb unit
h, in.	15	15	24
b_1, in.	36	36	18
A_c, sq in.	540	540	432
I, in.	10,125	10,125	20,736
e, in.	—	1.5	—

TABLE 3—PROPERTIES OF HIGH TENSILE STEEL TIE RODS

Bar diameter, in.	Suggested prestressing force, lb		Equivalent torque, ft-lb
	Initial	Final	
½	12,500	10,850	90
⅝	20,000	17,250	180
¾	29,000	25,600	320
⅞	37,000	32,400	470
1	49,000	42,500	710
1⅛	58,000	50,800	960
1¼	74,000	64,500	1350

Bending moments, M_c in precast curb units in longitudinal direction

The maximum moment in the curb due to four equal wheel loads is shown by the curve of Fig. 4. The group of wheel loads is placed so that the center line of the outside wheel is 2 ft from the face of the curb and the successive wheel distances are 6 ft, 4 ft, and 6 ft. One diaphragm is used ($m = 1$) and the edge stiffness ratio equals 2. The moment according to the formula $M_c = 0.01\ Pa$ given by the AASHO specification is also shown.

NUMERICAL EXAMPLES

In the first example, the stiffness of the curb unit is twice the stiffness of an interior slab unit ($H = 2$). One central diaphragm is used ($m = 1$). The second example also has one diaphragm but curb and slab units are of equal stiffness ($H = 1$). The third example is a bridge built of precast units placed side by side without transverse connection ($m = 0$).

Example 1

Clear width of roadway = 24 ft
Span of bridge, a = 20 ft
Number of precast slab units = 8
Number of precast structural curb units = 2
Spacing of slab units, b_1 = 3 ft
One diaphragm at midspan, m = 1
Loading = H20-S16-44 and 30 percent impact allowance

Properties of members are shown in Table 2.

Bending moments and stresses in center of slab in longitudinal direction:

Due to weight of slab

$$M_x = \tfrac{1}{8}\, pa^2 = \tfrac{1}{8} \times 188 \times 20^2 = 9400 \text{ ft-lb}$$

Due to four wheel loads and impact: from Fig. 2 for a = 20 ft, H = 2 and m = 1

$$M_x = \text{CP} = 0.96 \times 16{,}000 \times 1.30 = \underline{20{,}000 \text{ ft-lb}}$$
$$\overline{29{,}400 \text{ ft-lb}}$$

with d = 13.5 in., n = 12, f_s = 18,000 psi, f_c = 960 psi, $R = 29{,}400/13.5^2 = 162$, p = 0.0103

$A_s = 13.5 \times 12 \times 0.0103 = 1.67$ sq in. Use 1-in. diameter mild steel bars at 5½ in. on center.

Bending moment and stresses in transverse central diaphragm:

Due to four wheel loads and impact: from Fig. 3 for $a = 20$ ft, $H = 2$ and $m = 1$

$$M_y = CPb_1 = 0.63 \times 16{,}000 \times 1.30 \times 3.0 \times 12 = 473{,}000 \text{ in-lb}$$

Effective prestress force produced by three 1-in. diameter high tensile tie rods (Table 3) is $F = 42{,}500 \times 3 = 127{,}500$ lb and

$$\begin{aligned} M_F &= Fe = -127{,}500 \times 1.50 = -191{,}000 \text{ in.-lb} \\ M_y &= \qquad\qquad\qquad\quad +473{,}000 \text{ in.-lb} \\ \textstyle\sum M &= \qquad\qquad\qquad\quad +282{,}000 \text{ in.-lb} \end{aligned}$$

$$f_b = \frac{F}{A_c} + \frac{M}{Z} = -\frac{127{,}500}{540} + \frac{282{,}000 \times 7.5}{10{,}125} = -27 \text{ psi}$$

$$f_t = \frac{F}{A_c} - \frac{M}{Z} = -\frac{127{,}500}{540} - \frac{282{,}000 \times 7.5}{10{,}125} = -445 \text{ psi}$$

Bending moments and stresses in center of curb in longitudinal direction:

Due to weight of curb

$$M_c = \tfrac{1}{8}\, \varsigma a^2 = \tfrac{1}{8} \times 450 \times 20^2 = 22{,}500 \text{ ft-lb}$$

Due to four wheel loads and impact: from Fig. 4 for $a = 20$ ft, $H = 2$ and $m = 1$

$$M_c = \tfrac{1}{4}\, CPa = \tfrac{1}{4} \times 0.492 \times 16{,}000 \times 1.30 \times 20 = \underline{51{,}000 \text{ ft-lb}}$$
$$73{,}500 \text{ ft-lb}$$

with $d = 22$ in., $n = 12$, $f_s = 18{,}000$ psi, $f_c = 760$ psi, $R = (73{,}500 \times 12)/(18 \times 22^2) = 101$, $p = 0.0063$

$A_s = 0.0063 \times 22 \times 18 = 2.50$ sq in. Use three 1⅛-in. diameter mild steel bars at bottom of curb and say three ⅞-in. diameter at top.

Example 2

Data as in Example 1 but curbs not used as structural members.

Bending moment in center of slab in longitudinal direction:

Due to four wheel loads and impact: from Fig. 2 for $a = 20$ ft, $H = 1$ and $m = 1$

$$M_x = CP = 1.22 \times 16{,}000 \times 1.30 = 25{,}400 \text{ ft-lb}$$

Example 3

Data as before but no transverse connection between slab units.

Bending moment in center of slab in longitudinal direction:

Due to four wheel loads and impact: from Fig. 2 for $a = 20$ ft, $H = 1$ and $m = 0$

$$M_x = CP = 1.67 \times 16{,}000 \times 1.30 = 34{,}700 \text{ ft-lb}$$

CONCLUSION

A comparison of the examples shows that for a structure without transverse ties the live load moment is 34,700 ft-lb, but by the use of one central transverse connection and stiffened curbs this figure may be reduced to 20,000 ft-lb.

By adopting the method of construction outlined, a considerable saving over conventional practice can be achieved.

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Proportioning Concrete Mixtures Using Fly Ash*

By C. E. LOVEWELL† and GEORGE W. WASHA‡

SYNOPSIS

Results of compressive strength tests of concrete are shown comparing concrete mixtures containing Chicago fly ash with mixtures made without fly ash. The straight portland cement mixes contained from 4 to 6 bags of portland cement per cu yd of concrete and varied in half-bag increments. The mixes containing fly ash were proportioned to have approximately the same early strengths as the comparable straight portland cement mixes. To accomplish this the cement content could not be reduced by more than 1 bag per cu yd of concrete, and the fly ash added had to exceed the amount of cement removed. The amount of fly ash needed to replace each pound of cement removed increased as the richness of the original straight portland cement mix decreased. Data show that existing concrete technology can serve as a guide in proportioning concrete mixtures containing fly ash and that results can be predicted with reasonable accuracy.

INTRODUCTION

Many researchers[1,2,3,4] have shown that any percentage replacement of portland cement by fly ash on an absolute volume basis has resulted in lower strengths up to about 28 days but in greater strengths at 6 months and beyond when less than 30 percent of the cement was replaced. In some types of work, such as mass concrete, this pattern of strength gain is satisfactory. However, paving and structural concrete frequently are specified with a 14-day minimum strength requirement and much of the ready-mixed concrete produced is required to meet a minimum 28-day strength. Consequently, for these uses, portland cement concrete mixes made with fly ash were proportioned to have about the same early strength properties as the comparable straight portland cement concrete mixes. Initial test results were obtained at several commercial testing laboratories and at the materials testing laboratory at Rose Polytechnic Institute. The final test program as reported in this paper was carried out at the materials testing laboratory of the University of Wisconsin.

Series A

The basic program Series A required tests of 360 compression cylinders (6 x 12 in.). Results showed that approximate strength equality during the period from 3 to 28 days was obtained if not more than 1 sack of cement was

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†Member American Concrete Institute, Sales Manager and Service Engineer, Chicago Fly Ash Co., Chicago, Ill.
‡Member American Concrete Institute, Head, Mechanics Department, University of Wisconsin, Madison, Wis.

TABLE 1—AVERAGE FLY ASH ANALYSES*

Fly ash	Fe_2O_3, percent	SiO_2, percent	Al_2O_3, percent	CaO, percent	MgO, percent	Loss on ignition	SO_3, percent	Alaklies, total percent	Fineness, Blaine, sq cm per g
A	23.7	43.0	19.9	5.7	1.4	2.0	2.7	0.7	3930
B	17.7	47.8	19.6	5.6	1.3	2.0	2.3	1.0	3250
C	19.1	41.8	19.4	5.2	0.7	3.0	1.9		3230

*Unit weight of fly ash = 75 lb per cu ft.

removed from the straight portland mix and if it was replaced by more than an equal volume of fly ash. The fly ash replacement was greatest in mixes with lowest cement content, and in the maximum case the cement removed was replaced by about twice as much fly ash on an absolute volume basis. On the basis of available evidence it would be expected that fly ash mixes having 28-day strengths equal to those of straight portland cement mixes would also have the other important desirable properties of good concrete such as workability, watertightness, finishing characteristics, flexural strength, and durability (assuming equal air entrainment).

Series B

Tests reported in Series B represent a rather limited program (compression tests of 126 cylinders) to determine if other brands of Type I portland cement or Type IS portland blast furnace slag cement or other types of Chicago fly ash caused any marked differences in behavior characteristics. No significant differences were obtained in these tests. The fact that Type IS cements may be satisfactorily used in fly ash concrete mixtures has been reported under job conditions by R. J. McAllister.[5] In his report of the construction of the Shippingport Atomic Power Station he stated that on the basis of weekly compressive strength averages of 6900 cu yd of concrete made with Type IS cement (517 lb cement per cu yd) the average 28-day compressive strength was 4048 psi. The comparable strength based on 7600 cu yd of concrete made with Type IS cement and fly ash (414 lb cement and 91 lb fly ash per cu yd) was 3954 psi. Both types of concrete were placed using the same ingredients over the same 10-week period. Additional tests at 45 and 90 days indicated that the compressive strength of concrete made with fly ash at those ages was better than that made without fly ash.

Series C

Series C tests were performed to obtain additional information for the nominal 6 sacks per cu yd gravel concretes made with and without fly ash. Effects of variable ratio of sand to total aggregate and of air entrainment on the compressive strength were investigated. A total of 108 compression specimens were required for this program.

MATERIALS

Three fly ashes, all from the Chicago area, and five Type I portland cements and one Type IS portland cement were used. The fly ashes and the average analyses given in Table 1 were provided by the Chicago Fly Ash Co. The

TABLE 2—PROPERTIES OF AGGREGATES

Sieve No.	Madison sand	Average percent coarser than sieve			
		Gravel		Limestone	
		No. 4—¾ in.	No. 4—1½ in.	No. 4—¾ in.	No. 4—1½ in.
1½	—	0	0	0	0
1	—	0	1	0	14
¾	—	2	30	3	27
⅜	—	58	76	84	81
4	5	97	97	98	96
8	19	100	100	100	100
16	31	100	100	100	100
30	43	100	100	100	100
50	74	100	100	100	100
100	96	100	100	100	100
Fineness modulus	2.68	6.57	7.04	6.85	7.18
Unit weight, lb per cu ft	116	104	106	98	101
Absorption, percent	0.3	1.8		0.6	
Specific gravity, saturated, surface dry	2.69	2.68		2.66	

cements were purchased in their original containers from dealers in the Chicago area. The limestone was purchased in the Chicago area, while the Madison sand and gravel were obtained from dealers in Madison, Wis. Information on grading, unit weight, absorption, and specific gravity of the aggregates is provided in Table 2.

PROPORTIONING

The information in Table 3 regarding proportioning of concrete mixes made with and without fly ash resulted from several years of field experience supported by many additional laboratory studies. The maximum amount of portland cement that could be removed from a 4-bag and a 6-bag mix, respectively, and the amount of fly ash needed to compensate for the early strength loss due to removal of cement were established. It was clearly demonstrated that it was possible to remove more cement from the 4-bag mix than from the 6-bag mix, and also that the 4-bag mix required as much as 75 percent more fly ash than the 6-bag mix in order to get the early strengths of these mixes essentially equal to those of the comparable mixes made without fly ash. After information for the two extremes had been determined, necessary changes for intermediate mixes were obtained on the basis of a straight line interpolation.

Data in Table 3 show that the cement factor in the mixes without fly ash varied from 4.0 to 6.0 in increments of 0.5 sack per cu yd. Both gravel and stone coarse aggregates were used and maximum sizes for each coarse aggregate were ¾ and 1½ in. The ratio of sand to total aggregate for each mix is also given. In proportioning the fly ash mixes, 94 lb of portland cement was removed from the 4-sack mix, 70.5 lb of portland cement was removed from the 6-sack mix, and quantities removed from intermediate mixes were essentially in accord with a straight-line variation between the extremes. The actual cement factor for the mixes made with fly ash varied from 3.0 to 5.25 sacks per cu yd. The removal of the portland cement was compensated

TABLE 3—MIX PROPORTIONS FOR CONCRETE OF SERIES A

Maximum size coarse aggregate, in.	Original mix		Chicago fly ash adjusted mix			
	Cement factor, sacks per cu yd	Sand ratio*	Portland cement removed, lb	Cement factor, sacks per cu yd	Chicago fly ash added, lb	Sand ratio*
Concrete made with gravel aggregate						
1½	4.0	0.40	94	3.0	150	0.38
¾		0.46			175	0.42
1½	4.5	0.39	88	3.56	138	0.37
¾		0.45			156	0.41
1½	5.0	0.38	83	4.12	125	0.36
¾		0.44			137	0.40
1½	5.5	0.37	77	4.68	113	0.35
¾		0.43			118	0.39
1½	6.0	0.36	70.5	5.25	100	0.34
¾		0.42			100	0.38
Concrete made with stone aggregate						
1½	4.0	0.44	94	3.0	150	0.42
¾		0.50			175	0.46
1½	4.5	0.43	88	3.56	138	0.41
¾		0.49			156	0.45
1½	5.0	0.42	83	4.12	125	0.40
¾		0.48			137	0.44
1½	5.5	0.41	77	4.68	113	0.39
¾		0.47			118	0.43
1½	6.0	0.40	70.5	5.25	100	0.38
¾		0.46			100	0.42

*Sand ratio $= \frac{\text{sand}}{\text{total aggregate.}}$

for by the addition of 150 to 100 lb of fly ash for mixes made with 1½-in. maximum aggregate, as the cement factor varied from 3.0 to 5.25 sacks per cu yd. With ¾-in. maximum aggregate the additions varied from 175 to 100 lb. Also the ratios of sand to total aggregate for the fly ash mixes with 1½-in. maximum aggregate were reduced by 0.02 below the values for the concrete mixes without fly ash, and reductions of 0.04 were made for the concrete with ¾-in. maximum aggregate. Greater reductions in sand-to-total-aggregate values were made in some mixes in Series C (shown in Table 7). The amount of water to obtain slumps of 4 to 5 in. was determined by trial. Mixes containing fly ash and ¾-in. maximum aggregate had about 3.8 gal. water per cu yd of concrete less than comparable mixes made without fly ash for the same slump, while the decrease for mixes with 1½-in. maximum aggregate was about 2.8 gal. per cu yd of concrete.

MANUFACTURE AND TESTING

Three rounds were made for each variable, and three cylinders (one for each of the three test ages) were made for each round. The air dried aggregate and cementing materials were first machine-mixed for ½ min, then water was added, and the mixing continued for an additional 4½ min. After

TABLE 4—CONCRETE MADE WITH GRAVEL AGGREGATE, SERIES A

Mixes A-J made with ¾-in. maximum size gravel, mixes K-U made with 1½-in. maximum size gravel. Aggregates, air dry. Cement A, Fly ash A, slump 4-5 in.

Mix	Nominal cement content, sacks per cu yd	Materials per cu yd					Ratio w†	Unit weight, lb per cu ft	Ultimate compressive strength, psi		
		Cement, sacks	Fly ash, lb	Fine aggregate, lb	Coarse aggregate, lb	Water,* lb	cement + fly ash by weight		1 day	3 days	28 days
A	4	3.98	0	1540	1805	331	0.79	150.1	273	786	2470
B	4½	4.47	0	1480	1815	331	0.70	149.8	355	1020	3090
C	5	5.00	0	1435	1820	327	0.62	150.2	482	1305	3750
D	5½	5.50	0	1390	1835	322	0.55	150.7	572	1595	4055
E	6	5.98	0	1340	1850	324	0.51	151.0	716	1910	4835
F	4	3.03	175	1405	1940	289	0.54	151.9	266	886	3135
G	4½	3.59	156	1360	1965	288	0.50	152.4	314	1080	3425
H	5	4.13	137	1310	1965	297	0.49	152.0	370	1345	3720
I	5½	4.68	118	1260	1970	301	0.47	151.5	417	1605	4015
J	6	5.25	100	1220	2000	308	0.45	152.4	499	1870	4190
K	4	4.02	0	1370	2045	308	0.71	152.0	371	1170	3095
L	4½	4.51	0	1310	2055	309	0.62	152.0	456	1345	3260
M	5	5.03	0	1275	2075	308	0.56	152.8	600	1575	4115
N	5½	5.51	0	1220	2080	305	0.51	152.6	729	1860	4570
O	6	5.99	0	1180	2090	300	0.46	153.2	782	2095	4940
P	4	3.03	152	1295	2110	276	0.54	152.8	242	1200	3295
R	4½	3.59	139	1250	2125	278	0.50	153.1	297	1280	3575
S	5	4.15	126	1205	2135	286	0.47	153.2	360	1545	3805
T	5½	4.70	113	1155	2140	287	0.44	153.2	444	1815	4145
U	6	5.25	100	1105	2150	288	0.41	153.3	582	1965	4460

*Total water added.
†w = net water = water added minus water absorbed by aggregates.

TABLE 5—CONCRETE MADE WITH STONE AGGREGATE, SERIES A

Mixes A-J made with ¾-in. maximum size stone, mixes K-U made with 1½-in. maximum size stone. Aggregates, air dry. Cement A, Fly ash A, slump 4-5 in.

Mix	Nominal cement content, sacks per cu yd	Materials per cu yd					Ratio w†	Unit weight, lb per cu ft	Ultimate compressive strength, psi		
		Cement, sacks	Fly ash, lb	Fine aggregate, lb	Coarse aggregate, lb	Water,* lb	cement + fly ash by weight		1 day	3 days	28 days
A	4	3.97	0	1660	1660	344	0.83	149.4	287	932	2970
B	4½	4.49	0	1605	1665	344	0.78	149.4	386	1185	3275
C	5	4.98	0	1555	1685	345	0.70	150.4	478	1450	4110
D	5½	5.48	0	1510	1705	341	0.63	150.9	606	1645	4415
E	6	6.00	0	1465	1725	340	0.58	151.5	803	1990	5270
F	4	2.99	175	1510	1775	308	0.64	150.2	246	860	2910
G	4½	3.56	156	1465	1795	310	0.60	150.5	290	1085	3340
H	5	4.10	136	1425	1815	314	0.57	151.1	353	1400	3835
I	5½	4.66	118	1380	1835	312	0.53	151.3	525	1740	4220
J	6	5.21	100	1330	1835	316	0.51	150.9	587	2030	4815
K	4	4.00	0	1485	1885	321	0.81	150.6	387	1085	3090
L	4½	4.49	0	1430	1895	310	0.70	150.5	481	1360	3625
M	5	5.01	0	1380	1910	314	0.63	150.9	606	1665	4180
N	5½	5.50	0	1330	1920	313	0.58	151.3	722	1910	4450
O	6	6.00	0	1285	1925	310	0.52	151.2	791	2115	5185
P	4	3.01	151	1410	1950	288	0.62	151.2	306	1235	3250
R	4½	3.57	138	1370	1965	287	0.57	151.7	329	1320	3550
S	5	4.13	125	1325	1990	288	0.53	152.3	400	1575	4345
T	5½	4.70	113	1280	2000	290	0.49	152.2	519	1870	4645
U	6	5.25	100	1230	2010	295	0.47	152.9	686	2170	5395

*Total water added.
†w = net water = water added minus water absorbed by aggregate.

mixing, the concrete was discharged and checked for slump, specific weight, and air content. When air-entraining agents were used they were added to the mixing water. Three 6 x 12-in. cylinders were then cast in waxed cardboard molds and covered with wet canvas. After 24 hr the molds were removed and the cylinders were placed in a moist closet at 73 F to test date. Prior to testing, both ends of the cylinders were capped with high strength plaster. Each reported strength value is the average of three cylinders.

TEST RESULTS

Data necessary for comparison of the concrete mixes (Series A) made with and without fly ash over a nominal range of 4 to 6 sacks per cu yd and for gravel and stone aggregates with maximum sizes of $\frac{3}{4}$ in. and $1\frac{1}{2}$ in. are given in Tables 4 and 5 and Fig. 1 and 2. The quantities of all materials used in each batch are shown in Tables 4 and 5 along with the compressive strength

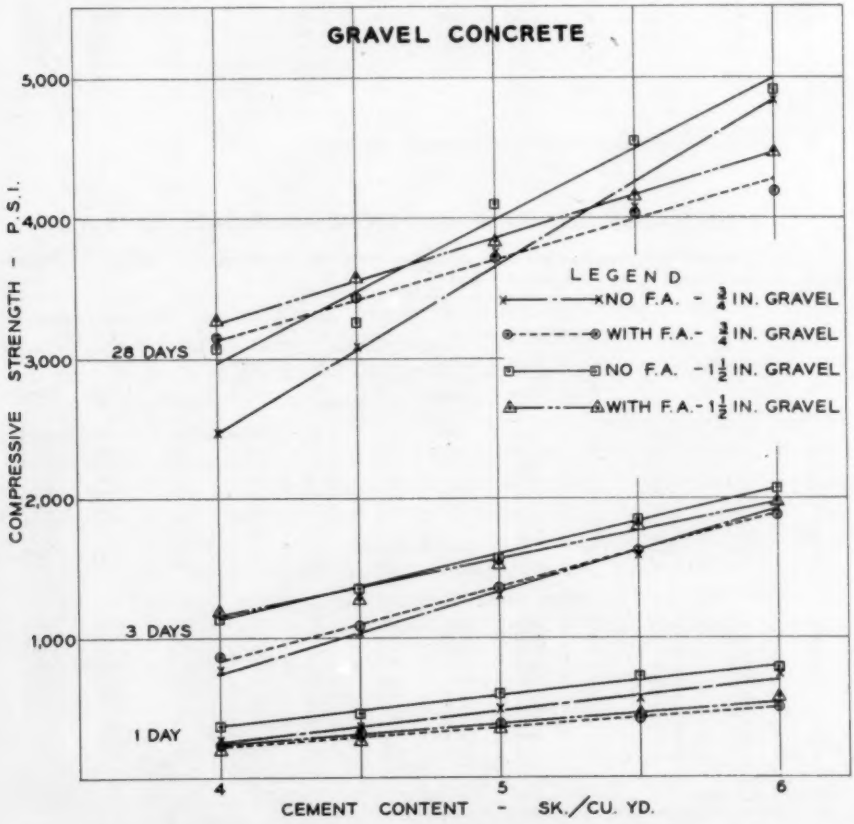

Fig. 1—Relation between compressive strength and cement content, gravel concrete

data. Curves in Fig. 1 and 2 indicate good parallelism between straight portland cement mixes and fly ash blends thereof at all bag levels and at all ages tested, with one exception. The 28-day strengths using gravel aggregates of both top sizes were greater for the fly ash blends than for their straight portland cement counterparts in the lean (4 and 4½ bags per cu yd) mixes and lower in the richest (6 bags per cu yd) mix.

Table 6 presents results for nominal 5-sack concretes made with and without fly ash for five different brands of Type I portland cement plus one brand of Type IS portland cement, and three different fly ashes. The six brands of cement were tested in concrete with and without the same fly ash and one brand of cement was tested with and without the three fly ashes, respectively. These tests were made to determine whether there was any noteworthy difference in relative behavior between various combinations. While it was apparent that there was a wide range in 1-day strengths between different

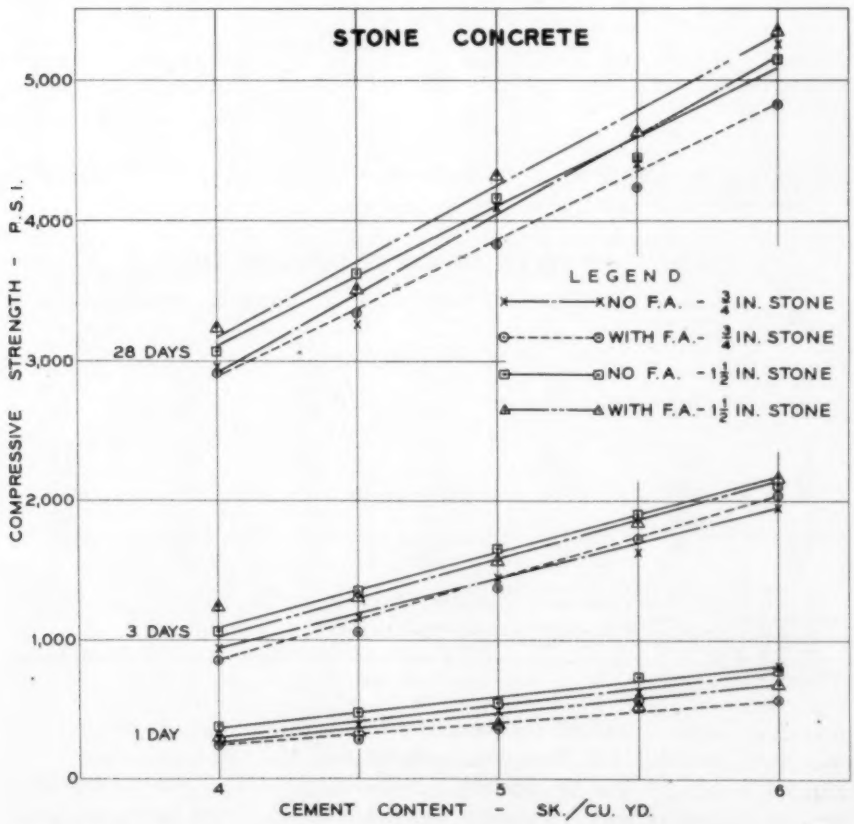

Fig. 2—Relation between compressive strength and cement content, stone concrete

TABLE 6—PROPERTIES OF CONCRETE MADE WITH DIFFERENT TYPES OF CEMENT AND FLY ASH, SERIES B*

Cement	Fly ash	Unit weight, lb per cu ft	Materials per cu yd					Ultimate compressive strength, psi, at		
			Cement, sacks	Fly ash, lb	Fine aggregate, lb	Coarse aggregate,† lb	Water,‡ lb	1 day	3 days	28 days
A Slag	—	151.3	4.99	—	1455	1845	319	315 (1.51)	1140 (1.01)	3510 (1.01)
A Slag	B	152.1	4.16	138	1305	1960	311	208	1125	3465
B	—	151.1	4.99	—	1455	1845	319	818 (1.35)	2155 (1.10)	4495 (1.09)
B	B	152.6	4.13	138	1310	1965	312	604	1950	4125
C	—	150.3	5.01	—	1445	1830	309	960 (1.25)	2110 (0.96)	3965 (1.04)
C	B	151.8	4.19	138	1305	1960	303	767	2190	3790
D	—	151.6	5.00	—	1460	1850	320	743 (1.48)	2140 (1.10)	4300 (1.08)
D	B	152.4	4.14	138	1310	1965	312	502	1945	3990
E	—	150.0	4.96	—	1445	1830	317	791 (1.20)	2210 (1.05)	4285 (1.09)
E	B	151.2	4.13	138	1300	1955	297	658	2100	3940
A	—	151.3	4.99	—	1455	1845	319	625 (1.57)	1650 (1.05)	3865 (1.06)
A	B	152.0	4.12	138	1310	1970	296	397 (1.25)	1580 (1.08)	3630 (1.07)
A	C	151.9	4.12	138	1310	1965	299	498 (1.66)	1535 (1.15)	3595 (1.05)
A	A	152.4	4.13	138	1315	1970	300	376	1435	3675

*Values in ( ) are ratios of strengths of concrete without fly ash to those of similar mixes made with fly ash.
†Gravel, ¾-in. maximum.
‡Total water added.

TABLE 7—PROPERTIES OF GRAVEL CONCRETE, SERIES C

Mix*	Cement content, sacks per cu yd	Fly Ash B, lb per cu yd	Ratio of sand to total aggregate	Ultimate compressive strength, psi, at		
				1 day	3 days	28 days
O	6.00	0	0.36	687	1710	4160
U	5.25	100	0.34	744	1690	4190
U1	5.25	100	0.31	757	1730	4170
U2	5.25	100	0.28	854	1805	4130
E	6.00	0	0.42	677	1740	4230
J	5.25	100	0.38	700	1630	4130
J1	5.25	100	0.35	677	1610	3880
J2	5.25	100	0.32	704	1615	4080
V†	6.00	0	0.34	676	1610	3430
W†	5.25	100	0.32	660	1395	3420
X†	5.25	100	0.29	674	1415	3430
Y†	5.25	100	0.26	630	1355	3240

*Mixes O, U, E, and J are the same as those described in Series A but were made at a later time and with a different shipment of Cement A.
Mixes O, U, U1, U2, V, W, X, and Y were made with 1½-in. gravel.
Mixes E, J, J1, and J2 were made with ¾-in. gravel.
†Mixes V, W, X, and Y made with an air-entraining agent respectively contained 4.4, 4.8, 4.5, and 4.5 percent of entrained air.

brands of portland cement, the respective blends thereof with fly ash followed the same pattern. The 3-day strengths showed the same wide range with parallel results for the fly ash blends. Variation in strengths at 28 days between brands of portland cement was much smaller, with the blends practically paralleling the straight mixes in this respect.

Strength results obtained in Series C are shown in Table 7. Within the limits of the tests performed the sand ratio had little effect on compressive strength. Generally strengths of the control mixes appeared to be about equal to the comparable mixes made with fly ash. With a given cement content the strengths of the concretes made with 4 to 5 percent entrained air were lower than comparable mixes without entrained air.

CONCLUSIONS

1. In order to obtain approximately equal compressive strengths at early ages, between 3 and 28 days, mixes made with fly ash must have a total weight of portland cement and fly ash greater than the weight of the cement used in the comparable straight portland cement mixes. The latter mixes will, however, contain from ¾ to 1 sack more cement per cu yd of concrete.

2. The maximum amount of fly ash should be used with lean concretes. With the materials used in these tests, 175 lb of fly ash was used to replace 1 sack of cement per cu yd of concrete in a nominal 4-sack mix, while 100 lb of fly ash was used to replace ¾ sack of cement per cu yd of concrete in a nominal 6-sack mix. The actual amounts will vary with the type of fly ash and aggregates used as well as with the richness of the mix.

3. The ratios of sand to total aggregate for mixes made with fly ash should be reduced 0.02 to 0.04 below those used in comparable straight portland cement mixes having equal early compressive strengths.

4. With relatively few check tests, mix proportions for concrete containing a given fly ash and a given set of aggregates can be developed that will give results which may be predicted with reasonable accuracy.

ACKNOWLEDGMENTS

The authors wish to thank the following for their help in carrying out this test program: L. M. Basford, M. C. Blake, J. C. Saemann, and N. H. Withey.

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U. S. Army photograph

Title No. 54-65

Military Personnel Records Center Built Without Expansion Joints*

By EARL B. COHN† and W. A. WALL‡

SYNOPSIS

One of the 20 largest buildings in the world is the Military Personnel Records Center in St. Louis, Mo. Construction of the main building (728 x 282 ft) without expansion joints is a significant design feature, and the case for omission of expansion joints is discussed in some detail. Cracking of floor slabs and building movement subsequent to construction are discussed.

This structure has demonstrated the feasibility of building large reinforced concrete structures without expansion joints; safety and satisfactory performance are possible if the structure is designed and construction controlled in conformity with recognized engineering principles.

INTRODUCTION

One of the 20 largest buildings in the world has been constructed by the federal government to house the records of all the military personnel of the United States. This building, the Military Personnel Records Center, was built in St. Louis, Mo., with the principal construction contract completed in 1956. Development of its design to house more than 38 million service records of the armed forces and to make them easily accessible to a working force of 4000 persons for as many as 56,000 daily references is an interesting story in itself, that has already been published (*Architectural Record*, Aug. 1956).

Cost experience indicated that the optimum economy of construction to house an activitity of this type containing 1,340,000 sq ft was a reinforced concrete warehouse-type structure, having six stories and basement. A six-

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†Structural Engineer, Kansas City District, Corps of Engineers, Kansas City, Mo.

‡Architectural Engineer, Kansas City District, Corps of Engineers, Kansas City, Mo.; Design Project Engineer for Military Personnel Records Center.

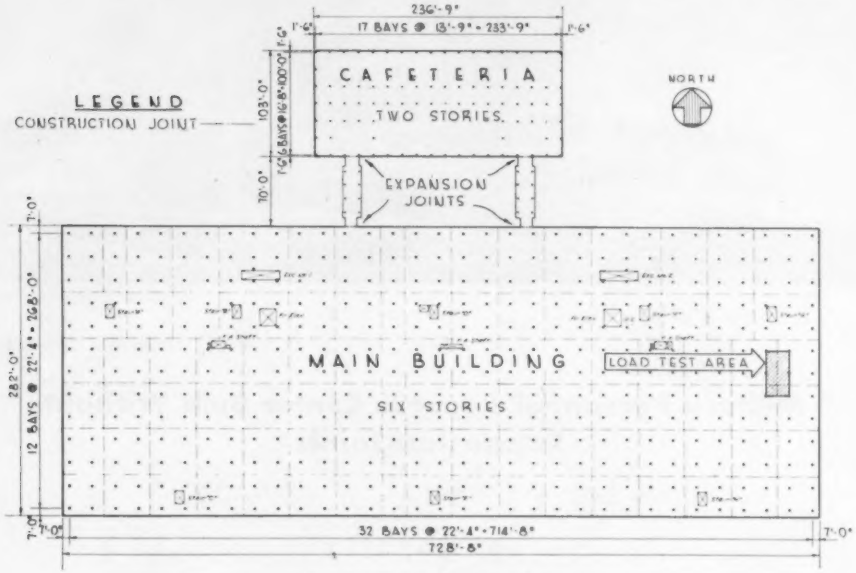


Fig. 1—Typical floor plan

story main building, 728 ft x 282 ft, and a two-story cafeteria, 236 x 103 ft, which is connected to the main building by two enclosed corridors, were ultimately developed. Construction of the main building without expansion joints is probably the most significant of the design features. Omission of expansion joints saved money, avoided an awkward plan condition, and minimized future maintenance problems. Expansion joints are used at both ends of the connecting corridors between the main building and the cafeteria wing (Fig. 1).

STRUCTURAL DESIGN

In general, the superstructure is standard skeleton construction of reinforced concrete, using conventional flat slabs with caps and drop panels on spirally reinforced square columns spaced 22 ft 4 in. on centers, with the first row of columns about 7 ft inside the exterior walls. Studies of various panel dimensions and slab thicknesses indicated that an 8 in. thick flat slab, 8 x 8-ft drop panels, and columns spaced 22 ft 4 in. on centers, having 5 x 5-ft caps, was the most economical design. The drop was made continuous between all outer columns on the perimeter of the building.

All floor slabs of the main building were designed for a live load of 200 psf, regardless of whether the area was used for storage or for office, thus allowing maximum flexibility in use of floor space. Column and footing designs were based on a live load of 180 psf on all floor areas. "Building Code Requirements for Reinforced Concrete (ACI 318-51)" was used as a basis for design.

Concrete for the project was mixed under inspection at a ready-mixed concrete plant and was made with Type I portland cement. A 5½-bag mixture with a water-cement ratio of 0.48 met the requirements for Class A concrete (3000 psi at 28 days).

Cast-in-place concrete pile footings and spread footings were considered in the preliminary design; the latter proved more economical. Differential and time settlement studies of a spread footing foundation were made from the results of laboratory tests. The building frame is sufficiently flexible to redistribute to quite an extent any irregular loading from differential settlement and strong enough as an entity to take appreciable differential settlement without damage.

Elimination of expansion joints

Expansion joints were not installed in the structural frame of the main building. The floor system design contemplated use of either a construction joint or a control joint with a weakened section at a distance of approximately 44 ft in both directions. The floor system was actually placed in a checkerboard pattern with an elapsed time of at least 48 hr before casting an adjacent unit. Under this procedure, all control joints became construction joints. Construction joints were made with the slab reinforcement continuous through the joint and with #4 dowels 3 ft long at 12 in. on center in the top of the slab. The length of placement in the grade beam walls was limited to 89 ft 4 in., with control joints at 22-ft centers. Control joints were placed in the grade beam walls midway between columns. They were made with 1¼ in. deep rubber strips in the outside face of the wall and ¾ in. deep redwood strips on the inside face. To provide a further weakened plane, alternate horizontal bars were cut at this control joint.

Before deciding to omit expansion joints from the building, the structural engineer made an extensive study of a number of large buildings constructed without expansion joints and of the movement of expansion joints in existing buildings. Measurements and checking of the movement of expansion joints in nine federal buildings in Washington, D. C., of various shapes and construction, designed by structural engineers of the Public Buildings Administration, showed as might be expected that the movement in expansion joints was less in air-conditioned buildings than in non-air-conditioned buildings, and also that the greatest movement generally takes place at the top floor. The movement of joints ranged from about ¼ in. in air-conditioned buildings to ½ in. for non-air-conditioned buildings.

The principal encouragement to the decision of the structural engineer to recommend the omission of expansion joints was obtained from inspection and study of the General Accounting Building in Washington, D. C. The General Accounting Building has seven stories and basement and an overall length of 638 ft; it was constructed without expansion joints, but control joints were placed throughout the floor and wall systems between alternate

columns or approximately 50 ft in each direction. No evidence of distress resulting from the omission of expansion joints was found.

The main building of the Military Personnel Records Center has an 8½ percent greater diagonal dimension than the General Accounting Building, but otherwise the factors considered are, in general, favorable to omission of expansion joints in the records center building. Both are flat slab construction and the heights are comparable. The records building is a perfect rectangle; the accounting building is not. The live load of the records center building is greater, resulting in sturdier construction. St. Louis and Washington, D. C., have substantially the same temperature differential. In rationalizing the case for the omission of expansion joints in the records center building, the structural engineer, William C. E. Becker, stated:

> Any attempt to logically explain the fact that there is no expansion distress in the accounting building leads to the tentative conclusion, at least, that the effects of shrinkage and a rise in temperature tend to offset each other.
>
> An examination of test data pertaining to shrinkage or volume change resulting from the setting of cement and the loss of water shows that the coefficient of shrinkage varies between 0.0003 and 0.0008, depending on many factors. The coefficient of expansion or contraction due to temperature change is about 0.0000055 per deg F.
>
> Assuming the coefficient of shrinkage to be 0.0003, hardening of the concrete will endeavor to cause a contraction equivalent to a drop in temperature of about 55 F. Correspondingly, a higher coefficient of shrinkage will cause the equivalent of a greater drop in temperature. Shrinkage prestresses reinforcing steel.
>
> This comparison indicates that shrinkage produces a deformation that is equivalent to the "absorption" of a certain amount of thermal deformation. In other words, after shrinkage at a coefficient of 0.0003, the temperature would have to rise 55 F in order to bring the deformation back to what it was before shrinkage began.
>
> If the frame is broken up, as it were, between control joints as was done in the accounting building and shrinkage is localized after a fashion, it seems that one could go on indefinitely without an expansion joint because a rise of 55 F or more in temperature is rare after a reinforced concrete frame is cast.
>
> In any case, it is reasonable and proper, in the opinion of the structural engineer, to conclude that experience with the Military Personnel Records Center with no expansion joints will be no worse than it is with the accounting building. There is some indication that it should be better since the records center building is a perfect rectangle, and the distance between control joints will be less (44 ft 8 in. as compared with 50 ft).
>
> That the effect of shrinkage substantially counteracts the effect of a rise in temperature in a building frame must be true, for any attempt to provide in the basement corner columns of buildings say 300 x 150 ft enough strength to resist the forces equivalent to those that would cause temperature elongations, will result in ridiculous sizes, compared with what is being used in such buildings with no distress. In other words, there are many buildings of such representative size (approximately 300 x 150 ft) where any consideration of temperature stresses in basement columns was neglected and they have shown no distress that can be traced to the omission of an expansion joint. Further, the structural engineer knows of no attempt in the construction of such buildings to control the location of shrinkage cracks as was done in the construction of the accounting building.

The structural engineer's arguments continued:

> Suppose the General Accounting Building had been built without control joints. In that case nature would have provided its usual unsightly and irregular shrinkage cracks,

"absorbing" the equivalent of a certain temperature drop. Beyond that there would be no further expansion until the "absorbed" stresses were relieved by a rise in temperature. Apparently, the accounting building frame has not been subjected to a rise in temperature of such magnitude as to completely eliminate the effect of shrinkage and consequently damage the architectural clothing. An examination of the exterior walls revealed no distress. Because of the success of the elimination of expansion joints in the accounting building it is confidently believed there should be corresponding success and satisfaction in eliminating expansion joints in the Military Personnel Records Center.

What has been done in the accounting building in providing no expansion joints corresponds to work in other fields of construction. In "A Discussion of Jointing Practice in Concrete Pavements" prepared by the technical staff of the Portland Cement Association, dated July 1939, shrinkage is shown conclusively to be more important than expansion. This discussion concludes that practically all expansion joints in road paving can be safely eliminated if contraction joints at proper intervals are provided and maintained. Casual observation of modern concrete highway construction reveals that the practical application of this conclusion has been successful and satisfactory.

Modern railroad construction has eliminated many rail expansion joints by localizing and safely resisting temperature stresses. The same is true for streetcar rail construction in paved streets.

Cracking in floor slabs

A somewhat disconcerting factor in the construction of the building was the appearance of considerable cracking in the floor slabs. The cracks were more numerous and wider on the second floor than elsewhere. These cracks, varying from hairline to 1/16 in. wide, do not appear to extend through the slab. Cores taken through the cracks indicate that they are closed on the under side. Fig. 2 shows the pattern of cracks visible on a part of the second floor. The crack pattern is most severe in a direction normal to the long dimension of the building.

The symmetrical crack pattern indicates that the cracks result from contraction of the floor slabs and from resistance to this contraction by the footings. The cracks are less prominent in the upper floors, probably because the footings offer less resistance to contraction there. Temperatures during concrete placement ranged from 20 to 108 F. The second-floor slab was cast during the summer of 1954 when temperatures of 100 F and over were common. This is considered a contributing factor to the cracking. Some of the crack locations can be attributed to the discontinuity or change in the amount of reinforcement in the top and bottom of the slabs. Since the building has been enclosed and maintained at a fairly constant temperature, the cracks in the floor are gradually closing up.

Load test

Although there was no serious doubt of the ability of the structure to carry the design load, it was considered desirable to conduct a load test on a floor section to determine its structural strength and to allay any doubts regarding the adequacy of the building to perform its function in safety.

The load test was performed in accordance with the ACI Building Code. Two panels were loaded with concrete blocks with load stages of 200, 250,

300, 350, and 400 psf. The loads were left in place for 24 hr prior to measuring deflections and increasing the load. Fig. 3 shows the deflections, which were less than one-half those permitted by the ACI Code. The residual deflection, which varied from 0.00 to 0.04 in. upon unloading, was considered negligible and no evidence of distress or further opening of cracks resulted from the test.

Building movement

Provision was made in the construction contract for the installation of bench marks on first story columns to permit periodic measurement of vertical

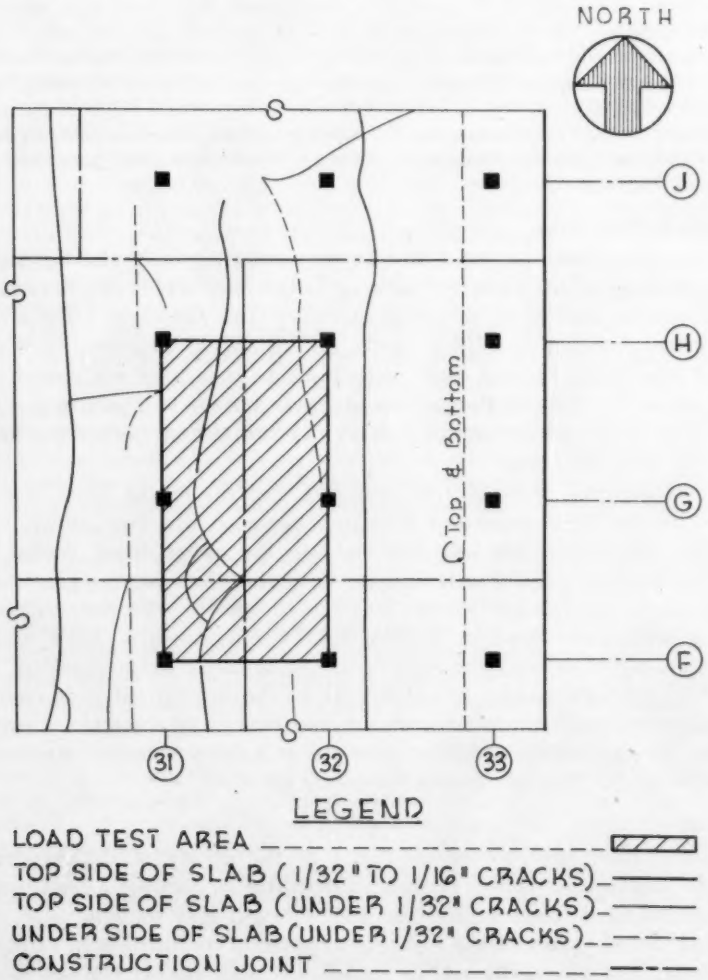
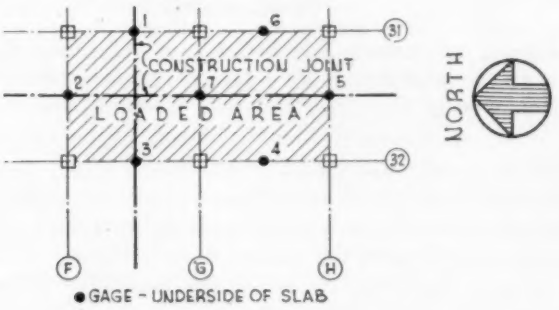


Fig. 2—Partial second floor plan showing crack patterns

movement of the building. Markers were also placed in the concrete frame to permit measurement of the changes in size of the building with temperature. Base markers consist of stainless steel plates cast flush with the exposed edges of the floor and roof slabs near the ends and at several intermediate points of the building. Bronze bars were placed in the roof slab to project above the roof and are at the perimeter of the building at intervals of about 100 ft. These markers provide a means of convenient, accurate measurement. Measurements to date, taken during extended periods of hot and cold weather, do not indicate any abnormal results.



SECOND FLOOR PLAN

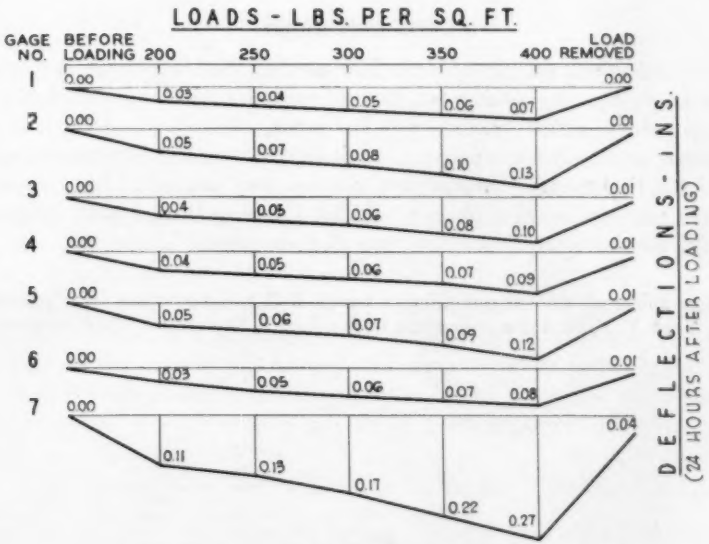


Fig. 3—Measured deflections during load test

Foundation settlement that has taken place is much less than the theoretical settlement and consequently differential settlements to date have been insignificant, although full live loading of the building has not taken place. The weight of excavation removed prior to placement of foundations, in some instances, exceeded the loads to be superimposed and therefore care was exercised in placing bench marks so that rebound in the foundation could be measured. The maximum measured rebound has been 0.014 ft and the greatest measured settlement has been 0.061 ft. These amounts are so small that if construction procedures, drying out of the foundation, and other factors are considered, no particular significance can be discerned.

CONCLUSIONS

In considering all factors that could have influenced the extent of the cracking, it was concluded that little, if any, beneficial effect would have been obtained if two expansion joints had been used. In the design of a similar structure, it is recommended that reinforcement be extended in the first two or three floor slabs above grade, regardless of structural requirements, to assure continuity beyond the pattern of maximum cracking. Also avoiding excessively high temperatures during placement and the initial curing should further minimize cracking tendencies.

This structure clearly demonstrates the feasibility of constructing safe, satisfactory reinforced concrete structures of considerable size without the use of expansion joints, if the structure is designed and construction controlled in conformity with recognized principles of engineering.

ACKNOWLEDGMENTS

This building was designed for the Kansas City District, Corps of Engineers, Kansas City, Mo., by Associated Architects and Engineers, a St. Louis, Mo., group which included: architects, Hellmuth, Yamasaki, and Leinweber; mechanical and electrical engineer, John D. Falvey; and structural engineer, William C. E. Becker. Information, advice, and assistance have been furnished the authors by Mr. Becker, and by Louis G. Feil, Chief, Engineering Division, Kansas City District, Corps of Engineers.

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Title No. 54-66

Creep and Creep Recovery of Concrete under High Compressive Stress*

By A. M. FREUDENTHAL† and FREDERIC ROLL‡

SYNOPSIS

Four series of tests studied creep and creep-recovery of concrete under sustained compressive stresses varying between approximately 15 and 65 percent of the 28-day compressive strength. Test specimens, loaded at 28 days, were cylinders 10 in. high, 3 and 4 in. in diameter, made with four different mixes. The creep tests were conducted under conditions of controlled temperature and humidity. Shrinkage of unloaded control specimens in the same environment was recorded so that the actual creep curves (total time-dependent deformation minus shrinkage) could be obtained.

Supplementary compression tests were conducted to determine the effect of sustained load on strength and modulus of elasticity of the concrete.

To reproduce and represent the observed creep and creep-recovery curves, a mechanical model was introduced consisting of four elements, each representing a specific type of contribution to total creep. Model constants were evaluated and their variation with respect to mix and applied stress determined.

Using creep equations derived from the model, creep was predicted for four stress levels of each mix of Series IV. The equations were also used for evaluating stress relaxation from various stress levels.

INTRODUCTION

The study of creep of plain concrete has been generally conducted at stress levels that were low compared to the ultimate compressive strength of the concrete. At such stress levels the creep was found to be directly proportional to the applied stress.[1-10] However, in view of the generally recognized nonlinearity between stress and creep rate at high stresses, none of the creep equations so far proposed can be applied at stresses higher than 20 to 30 percent of the ultimate compressive strength, the level where nonlinearity between stress and creep becomes apparent.[11,12]

The series of creep tests conducted at Columbia University were designed to study the effect of high stresses, to determine the response of various concrete mixes when subjected to the same specific stress levels (percentages of the 28-day ultimate compressive strength, f_c') to study the creep-recovery and the response of concrete to a history of loading-unloading cycles, and to

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†Professor of Civil Engineering, Columbia University, New York, N. Y.

‡Member American Concrete Institute, Associate Professor of Civil Engineering, University of Pennsylvania, Philadelphia, Pa.; formerly Associate in Civil Engineering, Columbia University.

devise a mechanical model which would reproduce the observed creep and creep-recovery curves, and thus make it possible to predict the creep and creep-recovery for various concretes subjected to any arbitrary stress sequence. No attempt was made to study the effect of the age of the concrete at the time of loading, nor the effect of environment; also, no attempt was made to study systematically the effect of the size of test specimen on the observed creep. Although effects of these variables are significant, their introduction would have required a prohibitive number of test replications.

DESCRIPTION OF TESTS

Materials

Concrete consisted of Type I, portland cement and common sand and gravel supplied by local, commercial dealers. The first two test series as described below were made with Saylor's portland cement; the third and fourth test series were made with Lone Star portland cement.

The sand had a fineness modulus of 2.08; the gravel with a nominal size range of ¾ to ¼ in. had a fineness modulus of 6.80, and the combined aggregate a fineness modulus of 5.10. For all mixes, the ratio by weight of sand to gravel was 1:1.5.

Three concrete mixes, 1:1, 1:6, and 1:2.5 (cement to total aggregate by weight) were used in each test series. A fourth mix, 1:4, was used in Series IV.

The 1:1 mix represented an essentially viscous body (cement paste) in which the aggregate is "floating"; the 1:6 mix was intended to represent an essentially solid mass of aggregate, the voids of which are filled by the viscous cement paste; the 1:2.5 mix was to represent an intermediate concrete, possibly the richest concrete mix still used in a structure.

Throughout this report each mix will be designated by the ratios indicated above. Mix proportions are shown in Table 1.

TABLE 1—DESCRIPTION OF TEST MIXES

Mix designation	Cement:sand:gravel (by weight)	Water-cement ratio		Volume concentration of aggregate*
		By weight	By absolute volume	
1:1	1:0.4:0.6	0.306	0.964	0.374
1:2.5	1:1.0:1.5	0.443	1.393	0.554
1:4	1:1.6:2.4	0.487	1.538	0.651
1:6	1:2.4:3.6	0.525	1.652	0.730

*Ratio of absolute volume of aggregates to absolute volume of concrete.

Curing and test environment

Curing conditions and test environment were identical for all mixes and tests. The fresh concrete was left in the cylinder molds for approximately 18 hr at ambient laboratory temperature and humidity. The molds were then stripped and the cylinders placed for 14 days in a moist-room at 100 percent relative humidity and approximately 70 F. Subsequently the cylinders were stored in an air-conditioned room at 70 F and 60 percent relative humidity.

After 13 days in the air-conditioned room (28 days after casting), some cylinders were moved to the laboratory for compression tests or placed in the loading rigs for the creep tests. The loaded cylinders in the creep rigs were then returned to the air-conditioned room where they remained until the completion of the creep tests, when they were again returned to the laboratory for removal from the creep rigs and subsequent compression tests. Additional cylinders of the same mix as tested in the creep rigs were kept in the air-conditioned room for compression tests on the last days of creep tests, and for shrinkage measurements.

Test specimens

The test specimens subjected to sustained load in the creep rigs were 10 in. high cylinders, some 3 in. in diameter, some 4 in. In addition to the 3 in. and 4-in. cylinders, 6 x 12-in. cylinders were also cast from the same batch to determine compressive strength and elastic modulus at 28 days and at the completion of the creep tests. Curing and storage conditions of the 6-in. cylinders were the same as for the 3 or 4-in. cylinders. Tests were started with 4-in. cylinders. The high compressive strength achieved necessitated a reduction to the 3-in. size to permit application of stresses up to 65 percent of compressive strength. The 6-in. cylinders served merely to correlate strength with standard tests.

Apparatus

The sustained loads were applied by means of specially designed compression spring type loading rigs shown in Fig. 1, similar to the rigs used by Davis and Davis,[12] Shank,[3] and Washa and Fluck.[13] The rigs were designed so that two cylinders could be loaded simultaneously in series. The 40,000 lb capacity springs had spring constants of approximately 10,000 lb per in., so that the small deformations of the test specimens due to creep had little effect upon the sustained load.

The deformation of the concrete cylinders (instantaneous, creep, instantaneous recovery, and time-dependent recovery) were determined by means of a 10-in. Whittemore strain gage. Readings were taken along three longitudinal gage lines symmetrically placed around the circumference of the cylinder.

To correct the observed deformations for shrinkage, special shrinkage rigs were constructed. Shrinkage readings were made by means of the 10-in. Whittemore strain gage along three longitudinal, symmetrically placed gage lines.

The temperature and relative humidity were controlled so that a nominal temperature of 70 F and a relative humidity of 60 percent were maintained throughout all tests. Temperature and humidity were recorded during tests so that any variations of creep and shrinkage due to temperature or humidity could be noted.

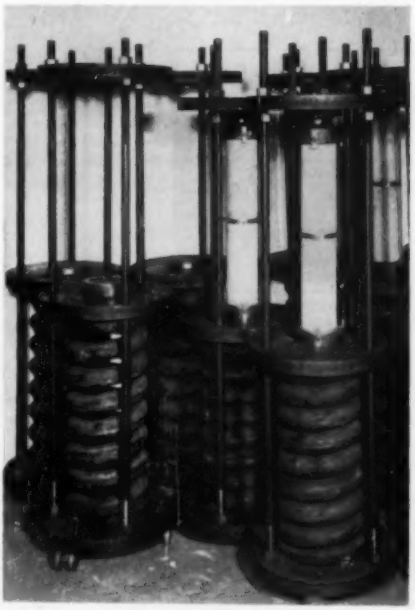

Fig. 1—Creep rigs in air-conditioned room

Test procedure

Each test series included the actual creep tests in which various sustained axial compressive loads were applied to the test cylinders, and deformations recorded, as well as compression tests on control cylinders and cylinders removed from the creep rigs at termination of the creep tests. Shrinkage readings on control cylinders were also recorded as part of the creep tests.

Although the series differed in certain details such as size of cylinders, loading, and duration of tests, the following procedures were common to all four series except as noted:

Creep tests—Before application of sustained test loads to cylinders in the creep rigs, an initial load of 500 lb was applied, except for Series I. Whittemore readings were then recorded at this initial load. Upon application of the test load, the instantaneous deformation due to the corresponding load increment was recorded. Instantaneous recovery was also recorded when the test load was reduced to 500 lb at completion of a

load cycle (see Series II) or at completion of a test. Periodic Whittemore readings were taken during loading and unloading periods, and the time-dependent deformations recorded.

Shrinkage readings were taken on cylinders of the same size and mix, and made from the same batch as those used in the creep tests. These shrinkage cylinders were stored in the same environment as those used in the creep test.

Supplementary compression tests—Compression tests to failure were performed on control cylinders on the first day of creep tests (28 days after casting), and on the last day of creep tests. These control cylinders were the same type and mix, cast from the same batch as used in the creep tests, and were stored in the same environment as cylinders used in the creep tests.

Compression tests on 6 x 12-in. cylinders cast from the same batch used to make control and creep test cylinders were conducted to determine the modulus of elasticity and

TABLE 2—DESCRIPTION OF TEST SERIES

Test Series	Description	Test duration, days	Mix	Specimen	Loading		
					Load, kips	Stress, psi	Percent of 28-day ultimate strength
I	*Sustained loads*	168	1:6	4 x 10 in.	40	3180	63
				4 x 10 in.	20	1590	31½
			1:2.5	4 x 10 in.	40	3180	51½
				4 x 10 in.	20	1590	26
			1:1	4 x 10 in.	40	3180	34
				4 x 10 in.	20	1590	17
II	*Periodically sustained loads:* Applied load sustained for 28 days followed by an unloading period (500 lb load) of 28 days; this sequence repeated three times. After third sequence, applied load sustained for additional 203 days.	371	1:6	4 x 10 in.	40	3180	62
				4 x 10 in.	20	1590	31
			1:2.5	4 x 10 in.	40	3180	56
				4 x 10 in.	20	1590	28
			1:1	4 x 10 in.	40	3180	35
				4 x 10 in.	20	1590	17½
III	*Sustained loads*	168	1:6	4 x 10 in.	40	3180	66
				3 x 10 in.	8.8	1245	20½
				3 x 10 in.	17.5	2480	40½
				3 x 10 in.	26.3	3720	61
				3 x 10 in.	35.0	4950	81
			1:2.5	4 x 10 in.	40	3180	54
				3 x 10 in.	9.4	1330	20½
				3 x 10 in.	18.7	2640	40½
				3 x 10 in.	28.1	3970	61
			1:1	4 x 10 in.	40	3180	41
				3 x 10 in.	12.9	1825	20½
				3 x 10 in.	25.8	3650	41
				3 x 10 in.	32.2	4550	51
				3 x 10 in.	38.7	5475	61
IV	*Sustained loads*	210	1:6	3 x 10 in.	8.5	1200	20
				3 x 10 in.	14.9	2110	35
				3 x 10 in.	21.2	3000	50
				3 x 10 in.	27.6	3900	65
			1:4	3 x 10 in.	9.0	1270	20
				3 x 10 in.	15.7	2220	35
				3 x 10 in.	22.5	3180	50
				3 x 10 in.	29.2	4130	65
			1:2.5	3 x 10 in.	9.8	1380	20
				3 x 10 in.	17.10	2420	35
				3 x 10 in.	24.4	3450	50
				3 x 10 in.	31.7	4480	65
			1:1	3 x 10 in.	12.6	1780	20
				3 x 10 in.	22.0	3110	35
				3 x 10 in.	31.5	4450	50
				3 x 10 in.	40.0*	5650	64

*Capacity of test rig.

compressive strength on the first and last days of creep tests. These cylinders were stored in the same environment as those used in the creep tests.

Compression tests to failure were performed on all creep test cylinders immediately after removal from the rigs at the completion of creep tests.

The moduli of elasticity were determined on the 6 x 12-in. cylinders and on the cylinders in the creep rigs. In both cases the moduli were defined in terms of the unloading stress increment and the corresponding instantaneous strain recovery.

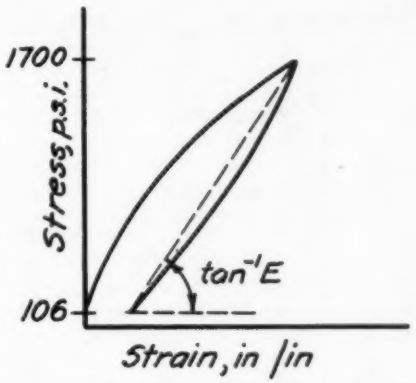

Fig. 2—Stress-strain diagram indicating method of determination of elastic modulus

Stress-strain curves in loading and unloading were recorded for the 6 x 12-in. cylinders by applying equal stress increments from an initial stress of 106 psi to a maximum stress of 1700 psi, and then reducing stresses in the same decrements to the final stress of 106 psi, as shown in Fig. 2. The maximum applied stress of 1700 psi was approximately 38, 30, and 19 percent of the 28-day compressive strength of Mixes 1:6, 1:2.5, and 1:1 respectively; these were approximately the same stress levels as those applied to half of the 4 x 10-in. cylinders used in the creep tests. (See Table 2.)

The moduli of elasticity of the cylinders in the creep rigs were determined by recording the instantaneous strain recovery caused by an unloading stress decrement, as discussed above under "creep tests," at the completion of a load cycle or at completion of tests.

The purpose and procedure of each test series is described below, and summarized in Table 2.

Test Series I—The 4 x 10-in. cylinders of mixes 1:1, 1:2.5, and 1:6 were loaded to full spring capacity (40 kips) and half spring capacity, and loads were sustained for 168 days.

The purpose was to study effects of variable concrete mixes upon creep at a given stress level, and the effect of that stress level upon creep for a given mix. The effect of sustained load upon strength and stiffness for the above parameters was also recorded.

Test Series II—Specimens, mixes, and magnitude of loads were the same as in Series I. The loads, however, were periodically sustained, being applied for 28 days and essentially removed (reduced to 500 lb) for another 28 days. This loading-unloading sequence was re-

TABLE 3—COMPRESSIVE STRENGTH AND MODULUS OF ELASTICITY, TEST SERIES I

Mix	Specimen	Loading of creep specimens		Compressive strength, psi						Modulus of elasticity,* million psi	
		Stress, psi	Percent of ultimate	Control specimens		Creep specimens	Ratios			Control specimens	
				28-day (1)	196-day (2)	196-day (3)	(2):(1)	(3):(1)	(3):(2)	28-day	196-day
1:6	4 x 10 in.	3180	63	5040	6210	6420	1.23	1.27	1.03	—	—
	4 x 10 in.	1590	31.5	5040	6210	6085	1.23	1.21	0.98	—	—
	6 x 12 in.	—	—	4510	5820	—	1.29	—	—	5.36	5.26
1:2,5	4 x 10 in.	3180	51.5	6160	7050	7410	1.14	1.20	1.05	—	—
	4 x 10 in.	1590	26	6160	7050	7250	1.14	1.18	1.03	—	—
	6 x 12 in.	—	—	6100	6900	—	1.13	—	—	4.58	4.68
1:1	4 x 10 in.	3180	34	9360	9310	10,420	0.99	1.10	1.11	—	—
	4 x 10 in.	1590	17	9360	9310	9410	0.99	1.01	1.01	—	—
	6 x 12 in.	—	—	8770	—	—	—	—	—	4.39	4.20

*Modulus of elasticity as defined in "test procedure."

TABLE 4—COMPRESSIVE STRENGTH AND MODULUS OF ELASTICITY, TEST SERIES II

Mix	Specimen	Loading of creep specimens*		Compressive strength, psi						Modulus of elasticity†, million psi					
				Control specimens		Creep specimens	Ratios			Control specimens		Creep specimens			
		Stress, psi	Percent of ultimate	28-day (1)	399-day (2)	399-day (3)	(2):(1)	(3):(1)	(3):(2)	28-day	399-day	56-day	112-day	168-day	399-day
1:6	4 x 10 in.	3180	62	5110	6370	6820	1.25	1.33	1.07	—	—	5.25	4.88	5.07	4.75
	4 x 10 in.	1590	31	5110	6370	6430	1.25	1.26	1.01	—	—	5.17	4.78	5.37	4.80
	6 x 12 in.	—	—	4920	6550	—	1.33	—	—	5.60	5.41	—	—	—	—
1:2.5	4 x 10 in.	3180	56	5670	6390	6810	1.12	1.20	1.07	—	—	4.30	4.26	4.26	3.22
	4 x 10 in.	1590	28	5670	6390	6200	1.12	1.09	0.97	—	—	4.37	4.26	4.26	3.62
	6 x 12 in.	—	—	5340	6760	—	1.27	—	—	4.39	4.45	—	—	—	—
1:1	4 x 10 in.	3180	35	9070	9690	9440	1.07	1.04	0.97	—	—	5.35	4.68	4.55	4.76
	4 x 10 in.	1590	17.5	9070	9690	10,340	1.07	1.14	1.07	—	—	4.16	4.52	4.78	3.82
	6 x 12 in.	—	—	8300	—	—	—	—	—	4.43	4.62	—	—	—	—

*Periodically sustained load. See description of test series. †Modulus of elasticity as defined in "test procedure."

TABLE 5—COMPRESSIVE STRENGTH AND MODULUS OF ELASTICITY, TEST SERIES III

Mix	Specimen	Loading of creep specimens		Compressive strength, psi						Modulus of elasticity,* million psi		
				Control specimens		Creep specimen	Ratios			Control specimens		Creep specimen
		Stress, psi	Percent of ultimate	28-day (1)	196-day (2)	196-day (3)	(2):(1)	(3):(1)	(3):(2)	28-day	196-day	196-day
1:6	3 x 10 in.	1245	20.5	6100	4740	6160	0.78	1.01	1.30	—	—	5.07
	3 x 10 in.	2480	40.5	6100	4740	5920	0.78	0.97	1.25	—	—	3.60
	3 x 10 in.	3720	61	6100	4740	6250	0.78	1.02	1.32	—	—	3.79
	3 x 10 in.	4950	81	6100	4740	†	0.78	†	†	—	—	†
	4 x 10 in.	3180	66	4840	5040	5520	1.04	1.14	1.10	—	—	4.45
	6 x 12 in.	—	—	4970	5240	—	1.05	—	—	5.56	4.87	—
1:2.5	3 x 10 in.	1330	20.5	6490	5760	6000	0.89	0.93	1.04	—	—	3.65
	3 x 10 in.	2640	40.5	6490	5760	6450	0.89	0.99	1.12	—	—	4.38
	3 x 10 in.	3970	61	6490	5760	5720	0.89	0.88	0.99	—	—	4.16
	4 x 10 in.	3180	54	5870	5880	6260	1.00	1.07	1.07	—	—	4.43
	6 x 12 in.	—	—	5740	6410	—	1.12	—	—	4.93	4.55	—
1:1	3 x 10 in.	1825	20.5	8940	7900	7960	0.88	0.89	1.01	—	—	4.10
	3 x 10 in.	3650	41	8940	7900	8090	0.88	0.90	1.02	—	—	4.57
	3 x 10 in.	4550	51	8940	7900	8040	0.88	0.90	1.02	—	—	4.66
	3 x 10 in.	5475	61	8940	7900	8290	0.88	0.93	1.05	—	—	4.74
	4 x 10 in.	3180	41	7750	7250	7830	0.94	1.01	1.08	—	—	4.26
	6 x 12 in.	—	—	—	8730	—	—	—	—	4.49	4.09	—

*Modulus of elasticity as defined in "test procedure." †Specimens failed during creep tests after 10 days.

peated three times. After the third sequence, loads were re-applied and sustained for an additional 203 days.

The purpose was to study effects of periodically sustained loads upon creep, strength, and stiffness, as well as the time-dependent elastic recovery upon unloading (creep-recovery).

Test Series III—Sustained loads of 20, 40, and 60 percent of the 28-day ultimate strengths of Mixes 1:1, 1:2.5, and 1:6 were applied to 3 x 10-in. cylinders for 168 days. In addition, a sustained load of 50 percent of ultimate strength was applied to a 3 x 10-in. cylinder of Mix 1:1 for 168 days, and a load of 80 percent of ultimate strength to a 3 x 10-in. cylinder of Mix 1:6 for 10 days, at which time an explosive failure occurred. A sustained load of full spring capacity (40 kips) was applied to 4 x 10-in. cylinders of the three mixes.

The purpose was the same as for Series I. Use of a 3 in. diameter specimen permitted the application of the same high stress levels for all three mixes and thus a more direct comparison of the effect of concrete mix upon creep for a given stress level. Use of at least three stress levels gave a clearer picture of the nonlinearity of the stress-creep relation. The 4-in. cylinder was used only for comparison with results of Series I.

Test Series IV—Sustained loads of 20, 35, 50, and 65 percent of the 28-day ultimate strengths of Mixes 1:1, 1:2.5, 1:4, and 1:6 were applied to 3 x 10-in. cylinders for 210 days.

The purpose was to compare theoretical creep, as predicted by the proposed concrete model and model constants for the various mixes, with the actual creep.

TEST RESULTS

Results of the supplementary compression tests, and the moduli of elasticity are shown in Tables 3, 4, 5, and 6. Except for Series IV, these tables give 28-day compressive strengths, the strength of unloaded control specimens at the completion of creep tests, and the strength of creep specimens after removal from the creep rigs.

Age-strength relationship

As expected, the effect of aging alone was to increase strength of the 4 x 10-in. and 6 x 12-in. cylinders for Mixes 1:6 and 1:2.5. The effect of aging on the rich 1:1 mix, however, was negligible; in some cases it showed a slight decrease in strength with age. The effect of aging on the 3 x 10-in. cylinders was to decrease the strength for all mixes as shown in Table 5. Although this differs from the other results, the observations are so consistent as to suggest that the specimen size may have some influence upon the aging properties of concrete. The relatively rapid drying of the small specimens may actually have a detrimental effect upon the strength of the concrete. Part of this effect may, however, be due to the fact that a different brand of cement was used in Series III than in Series I and II, as seen by the smaller increase in strength with age of 4 x 10-in. and 6 x 12-in. specimens in Series III, compared with the strength increase in Series I and II.

Sustained load effect on strength

The effect of sustained load upon compressive strength is comparatively small for the 4 x 10-in. cylinders. The 3 x 10-in. cylinders of Mix 1:6, however, showed an appreciable increase in strength due to the sustained loads. The 3 x 10-in. cylinders of Mix 1:2.5 showed a smaller increase, and those of Mix 1:1 a negligible increase. The amount and duration of sustained load seemed to have no effect upon the increase in concrete strength.

The effects of age and sustained load on compressive strength of larger cylinders agree essentially with the results of Washa and Fluck[13] for 6 x 12-in. concrete cylinders of various mixes and types of cements. Although the maximum stress applied by Washa and Fluck was only 25 percent of the ultimate strength for a period of 10½ years, whereas a maximum stress of 66 percent was applied to the 4 x 10-in. cylinders for periods of 196 to 399 days, the ratios of increase in strength due to sustained loads was remarkably close. The maximum increase was 12 percent for Washa and Fluck's tests, and 11 percent for the Columbia tests.

Modulus of elasticity

The effect of age alone upon the modulus of elasticity of the 6 x 12-in. cylinders, as previously defined, is negligible, or it may even cause a decrease in modulus. This directly contradicts the accepted belief of an increase in modulus of elasticity with age, when the modulus is defined in the conventional manner as the slope of a secant of the loading stress-strain curve. The moduli of elasticity of the 4 x 10-in. creep specimens of Series II, as defined—a function of the *instantaneous* recovery due to unloading—showed no significant effects of age or number of preceding load cycles. Nor did the magnitude of the sustained stress have any significant effect on the 4 x 10-in. or 3 x 10-in. cylinders (see Tables 4, 5, and 6).

The 28-day compressive strengths and moduli of elasticity of 6 x 12-in. cylinders, and the moduli of the creep specimens when unloaded after 210

TABLE 6—COMPRESSIVE STRENGTH AND MODULUS OF ELASTICITY, TEST SERIES IV

Mix	Specimen	Loading of creep specimens		28-day compressive strength, psi	Modulus of elasticity,* million psi	
		Stress, psi	Percent of ultimate		Control specimen 28-day	Creep specimen 210-day
1:6	3 x 10 in.	1200	20	6000	—	4.35
	3 x 10 in.	2100	35	6000	—	5.10
	3 x 10 in.	3000	50	6000	—	5.05
	3 x 10 in.	3900	65	6000	—	5.05
	4 x 10 in.	—	—	4990	—	—
	6 x 12 in.	—	—	4260	5.93	—
1:4	3 x 10 in.	1270	20	6370	—	4.20
	3 x 10 in.	2230	35	6370	—	4.85
	3 x 10 in.	3180	50	6370	—	4.76
	3 x 10 in.	4140	65	6370	—	5.03
	4 x 10 in.	—	—	5930	—	—
	6 x 12 in.	—	—	5670	5.21	—
1:2.5	3 x 10 in.	1380	20	6900	—	3.90
	3 x 10 in.	2420	35	6900	—	4.50
	3 x 10 in.	3450	50	6900	—	4.56
	3 x 10 in.	4480	65	6900	—	4.63
	4 x 10 in.	—	—	6900	—	—
	6 x 12 in.	—	—	5950	4.58	—
1:1	3 x 10 in.	1780	20	8900	—	4.33
	3 x 10 in.	3110	35	8900	—	4.61
	3 x 10 in.	4450	50	8900	—	5.05
	3 x 10 in.	5650†	64†	8900	—	4.81
	4 x 10 in.	—	—	8550	—	—
	6 x 12 in.	—	—	8270	4.68	—

*Modulus of elasticity as defined in "test procedure."
†Limited by capacity of test rig.

days of sustained load have been tabulated for Series IV for comparison with the first three test series.

Creep study results

Characteristic creep test results are presented in Fig. 3 to 9 (which also show the shrinkage of the controlled cylinders); creep is defined as the total axial strain due to a constant sustained stress, less the shrinkage strain, and less the initial instantaneous strain. The creep-recovery portions of Fig. 6 were obtained by subtracting the initial instantaneous strains from the total strain recovered in the concrete after removal of load.

The nonlinearity of the creep-stress relation is apparent in all cases, even for 4 x 10-in. cylinders of Mix 1:1 on which the maximum stress was only 34 percent of the ultimate strength. To illustrate this nonlinearity, the average creep was plotted as a function of the specific stress level (ratio of applied stress to the 28-day ultimate strength) with time as a parameter, for Series III as shown in Fig. 10 to 12. From these curves, it would appear that creep is a linear function of stress up to stresses which are approximately 20 to 26 percent of the ultimate strength of the concrete, but nonlinear for higher stresses. This limiting stress level appears related to time, being slightly less when the load has been sustained for only a short period, and slightly higher when the load has been sustained for a long time.

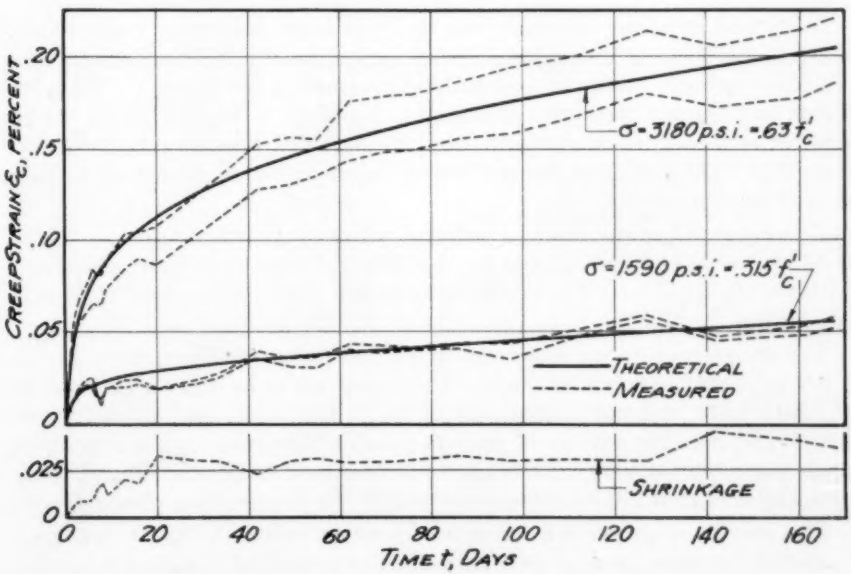

Fig. 3—Creep curves obtained with 4 x 10-in. cylinders of Mix 1:6 stressed respectively at 31.5 and 63 percent of 28-day compressive strength, compared with theoretical curves, Eq. (2b), Series I, (Test results from two cylinders in rig)

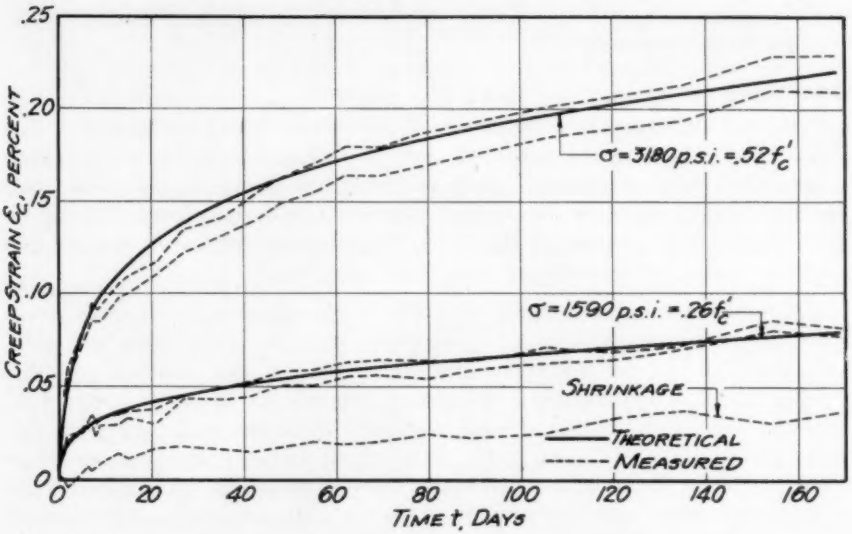

Fig. 4—Creep curves obtained with 4 x 10-in. cylinders of Mix 1:2.5 stressed respectively at 26 and 52 percent of 28-day compressive strength, compared with theoretical curves, Eq. (2b), Series I

Tests conducted by Glanville[11] on 1:1½:3 rapid hardening concrete indicated the limiting stress level to be approximately 22½ percent of the ultimate strength when the load had been sustained for 14 days. Tests by Davis and Davis[12] show that the creep is linear up to stresses which are 26 percent of the ultimate strength when the load had been sustained for 200 days and 1200 days, and showed definite nonlinearity at stresses of 40 percent of the ultimate.

When comparing the creep of different mixes subjected to the same stress level, it is seen in Series III and IV that Mix 1:2.5 has the largest creep, and Mix 1:6 the smallest. The nonlinearity of the creep due to high stresses is the highest for Mix 1:1.

The creep-recovery, as illustrated in Fig. 6 appears to be complete within a few days after removal of the stress, and appears to be a linear function of the previously sustained stress, regardless of the magnitude of that stress level. Nor does the number of preceding load cycles seem to have any effect upon the creep-recovery. Thus the total recovery, instantaneous and time-dependent, is at any time a linear function of the previously applied stress.

The results described are limited to tests on relatively small cylinders, loaded at the same age and for relatively short periods of time, and subjected to the same environmental conditions. Tests conducted under other conditions would lead to different quantitative results, and might even change the relationships between these results.

PROPOSED MECHANICAL MODEL; MODEL CONSTANTS

The nature of creep and creep-recovery has been discussed by several investigators, notably Shank,[3] Lorman,[4] Freudenthal,[14] Arnan, Reiner, and Teinowitz,[15] Ross,[6] and Cowan.[9] According to Freudenthal:

> Concrete is formed by an aggregation of loose grains (sand and aggregate) held together by a highly viscous liquid, the cement paste. The viscosity of this liquid increases with time, as a result of chemical changes within the structure (crystallization) until a complete crystalline network blocks all viscous deformation.
>
> Hardened cement paste is essentially a (nonlinear) Maxwell body of high viscosity and cohesion; the aggregates and sand form a noncohesive, granular mass, the resistance of which to irrecoverable deformation by shear (in this case identical with "gliding rupture") is the result of friction between the grains, which in turn is a function of the applied hydrostatic pressure.

Thus, the application of load causes an initial instantaneous deformation, part of which is perfectly elastic and is due to the response of the statistically isotropic rigid skeleton formed by the sand, gravel, and cement crystals in intimate contact, and part of which is inelastic, resulting in permanent set. This latter deformation is caused by the local destruction of cohesion between the aggregate particles and the cement paste (Freudenthal, and Arnan, Reiner, and Teinowitz). Removal of the load produces, therefore, an instantaneous recovery which is less than the initial instantaneous deformations; subsequent loading or unloading however causes deformations which are equal to the initial instantaneous recovery.

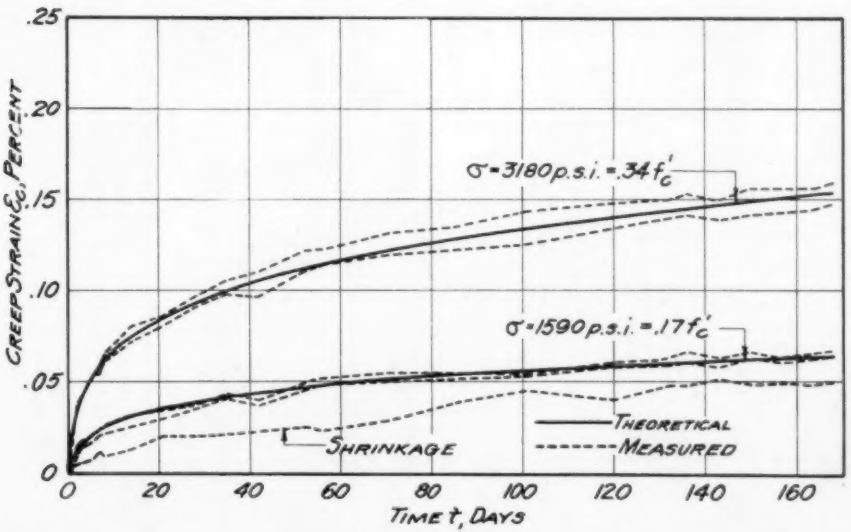

Fig. 5—Creep curves obtained with 4 x 10-in. cylinders of Mix 1:1 stressed respectively at 17 and 34 percent of 28-day compressive strength, compared with theoretical curves, Eq. (2b), Series I. Test results from two cylinders in rig

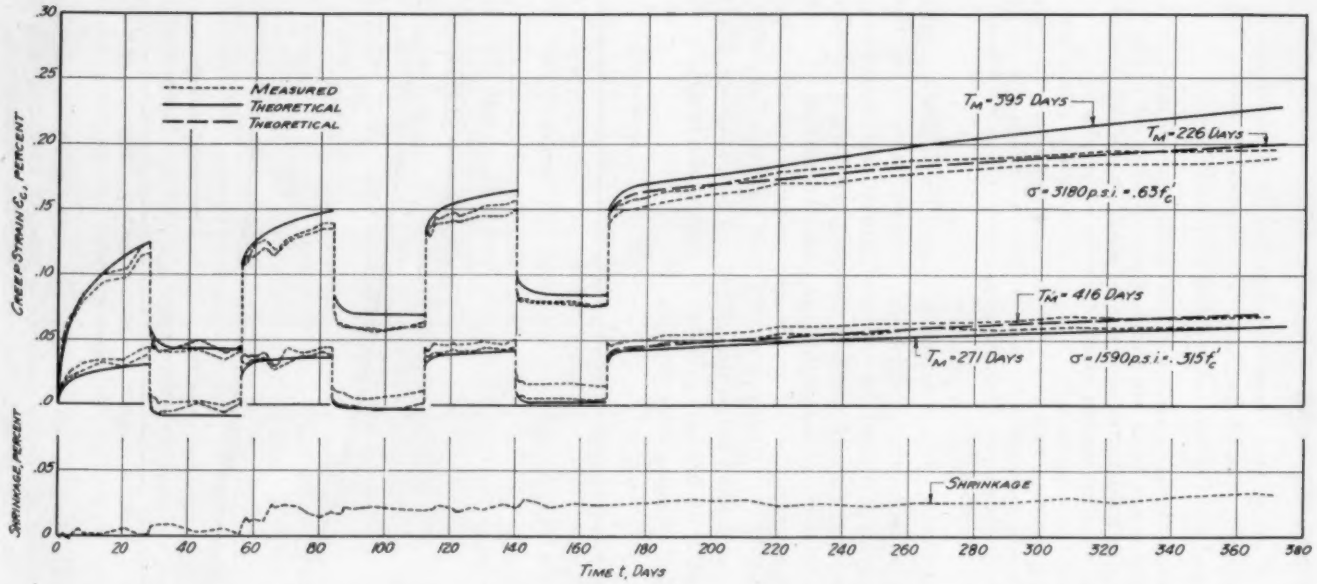

Fig. 6—Creep and creep-recovery curves obtained with 4 x 10-in. cylinders of Mix 1:6 stressed respectively to 31.5 and 63 percent of 28-day compressive strength, and periodically unloaded, compared with theoretical curves, Eq. (1b). The broken theoretical curve is obtained by adjustment of the parameter of long-time creep. Series II, test results from two cylinders in rig

Continued application of the load causes the concrete to creep, this creep being due to:

1. Viscous flow of the cement paste (Lorman, Freudenthal, etc.);
2. Consolidation due to seepage, or flow of adsorbed water from the cement gel due to applied pressure (Lorman and Ross);
3. Delayed elasticity due to the cement paste acting as a restraint on the elastic deformation of the skeleton formed by the aggregate and the cement crystals in the manner of "Kelvin's sponge" (Cowan, and Arnan, Reiner, and Teinowitz);
4. Permanent deformation caused by localized fracture.

The total creep increases at a decreasing rate due partly to the increase in viscosity of the cement paste as it crystallizes, partly to the completion of the delayed elastic and inelastic deformations, and partly to the termination of the seepage effect.

Upon removal of the load, the instantaneous recovery is followed by the delayed elastic recovery, or "creep-recovery." The deformations due to viscous flow and local destruction are permanent. The deformation due to

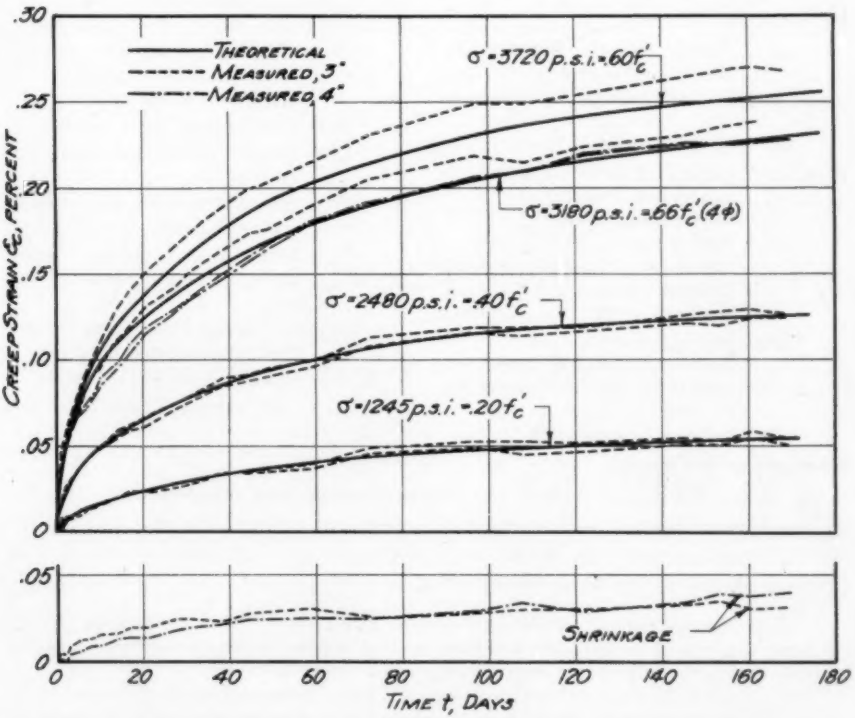

Fig. 7—Creep curves obtained with 3 x 10-in. cylinders of Mix 1:6 stressed respectively at 20, 40, and 60 percent of 28-day compressive strength, compared with curves of 4 x 10-in. cylinder stressed at 66 percent of compressive strength as well as with theoretical curves. Eq. (2b). Series III, test results from two cylinders in rig

seepage, however, may be completely recoverable, partially recoverable, or irrecoverable, depending upon environmental conditions.

When load is applied to the concrete the adsorbed water of the gel is subjected to a hydrostatic pressure gradient which expels this water from the gel toward the surface of the concrete. When the pressure gradient decreases sufficiently so that no more water is expelled, the seepage effect ceases and there is no further creep due to seepage. If the load is now removed, water on the surface of the concrete is free to return to the pore spaces of the gel, the gel expands, and creep due to seepage is fully recovered. If, however, the concrete surface is surrounded by a low relative humidity, the water expelled to the surface evaporates. If all of the expelled water evaporates before the load is removed, then removal of load is not accompanied by recovery, and the creep due to seepage is a permanent deformation. Partial evaporation before removal of load results in partial recovery after load removal. Thus, the only way in which complete recovery of the creep due to seepage can occur is to keep the concrete at 100 percent relative humidity.

Notation

φ_M = fluidity (inverse viscosity) coefficient of Element (a)
$\varphi_K, \varphi_1, \varphi_2$ = fluidity coefficients of Elements (b), (c), and (d) respectively
α_M = spring constant of Element (a)
$\alpha_K, \alpha_1, \alpha_2$ = spring constants of Elements (b), (c), and (d) respectively
x = displacement in model
P = force in model
ϵ = uniaxial strain in creep specimen
σ = uniaxial, constant stress in creep specimen
t = time
τ_M = relaxation time of Element (a)
τ_K, τ_1, τ_2 = retardation time of Elements (b), (c), and (d) respectively
b = dimensionless constant
C = constant with magnitude of unity, but units of time^{-1}
σ_o = stress limit of linear response
f_c' = ultimate compressive strength of concrete
σ_o/f_c' = critical stress ratio (where linear relationship ends)
C_V = ratio of absolute volume of aggregates to absolute volume of concrete
W_C = ratio of absolute volume of water to absolute volume of cement
W/C = water-cement ratio by weight
W/A = water-aggregate ratio by weight

Elements of the model

To reproduce and represent the observed creep and creep-recovery curves at the various applied stress levels by analytical expressions, and to identify the different components of the total deformation, a mechanical model was devised consisting of the following (Fig. 13):

1. A Maxwell element, consisting of a linear spring and of a dashpot which is nonlinear with respect to both force and time, coupled in series, which represents the long-time viscoelastic response in shear resulting in irrecoverable creep;
2. A Kelvin or Voigt element, consisting of a linear spring and dashpot coupled in parallel, which represents the viscoelastic interaction in shear of solid and fluid phases resulting in recoverable creep or "delayed elasticity";
3. A Kelvin element, nonlinear with respect to force, representing the short-time "consolidation" effects of irrecoverable pore-water motion (seepage) towards the surface and subsequent evaporation;

4. A Kelvin element, nonlinear with respect to force, representing the irrecoverable deformation due to initial, internal, partially destructive readjustments within the granular mass of the concrete.

These elements are coupled in series, so that the total deformation and deformation rate at any time can be represented by the sum of the deformations and deformation rates of each element. Elements (a) and (b) respond to increasing and decreasing forces, but Elements (c) and (d) respond to increasing forces only. The latter is necessary to reproduce the fact that neither the internal readjustment, nor the pore-water motion and drying out effects can generally be reversed by reversing the applied forms, although in the case of (c) a significant recovery might be expected on removal of forces, followed by release of the pore-water overpressure if evaporation at the surface were prevented, as in the case of imperviously covered or submerged concrete.

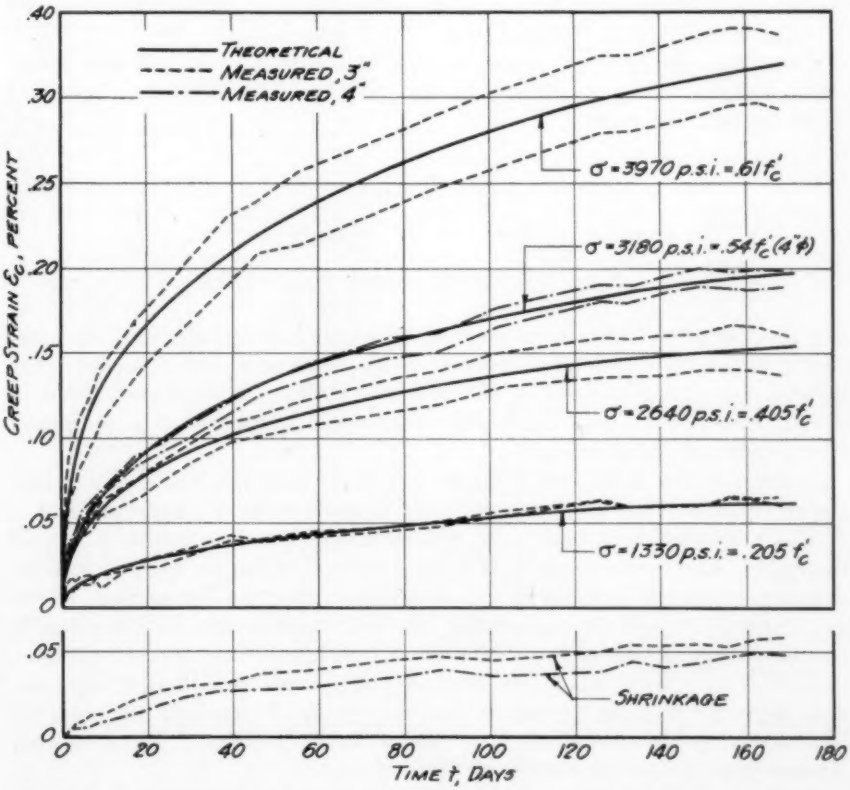

Fig. 8—Creep curves obtained with 3 x 10-in. cylinders of Mix 1:2.5 stressed respectively at 20.5, 40.5, and 61 percent of 28-day compressive strength, compared with curves of 4 x 10-in. cylinders stressed at 54 percent of compressive strength as well as with theoretical curves, Eq. (2b). Series III, test results from two cylinders in rig

To be consistent with the fact that equal amounts of creep-recovery occur for equal periods of zero load regardless of the number of previous loading-unloading cycles, as discussed earlier, it is reasonable to assume that the spring constant α_K and fluidity φ_K (inverse viscosity coefficient) of Element (b) remain constant with respect to time and force. For simplicity, and in view of the relatively short time that Elements (c) and (d) are effective, it is assumed that α_1, α_2, φ_1, and φ_2 are time-independent; but due to the response of seepage and internal destruction of cohesion, they must be force-dependent.

The spring constant α_M of Element (a) is both force and time-independent in accordance with the observed fact that equal instantaneous deformation (except for the initial instantaneous deformation) and recovery occurs regardless of the number of previous loading-unloading cycles. Since the viscosity of the cement paste changes as the paste crystallizes, and since the crystallization depends on time and, probably, pressure, the fluidity is assumed to be dependent upon time and load.

Identifying displacement x and force P in the model with uniaxial strain ϵ and uniaxial, constant stress σ in the creep specimen, the creep rate and creep at time t is given by

$$\dot{\epsilon}_c(\sigma,t) = C e^{-bt} e^{-t/\tau_M} + \sigma \alpha_K e^{-t/\tau_K} + \sigma \alpha_1 e^{-t/\tau_1} + \sigma \alpha_2 e^{-t/\tau_2} \quad \text{.....(1a)}$$

$$\epsilon_c(\sigma,t) = Ce^{-bt} T_M(1 - e^{-t/\tau_M}) + \sigma \alpha_K(1 - e^{-t/\tau_K}) + \sigma \alpha_1(1 - e^{-t/\tau_1}) + \sigma \alpha_2(1 - e^{-t/\tau_2}) \quad \text{.....(1b)}$$

If at a time $t = t_1$, the stress is removed, there is an instantaneous recovery, $\sigma\alpha_M$, followed by the delayed recovery of the reversible Kelvin model. The deformation of the dashpot of the Maxwell body, and the deformations of Kelvin Elements (c) and (d) which were assumed to respond to increasing forces only, are irrecoverable.

To simplify the evaluation of the six Kelvin constants, it was assumed that the retardation times τ_K and τ_2 were of the same order of magnitude, and τ_1 of a higher order, such that $\tau_1 = 10\tau_2 = 10\tau_K$. These assumptions seemed reasonable since creep-recovery and the deformation due to internal destruction are apparently completed in a short time, whereas the seepage effects and consequent surface evaporation probably take longer to complete. It was also assumed that a simple relation existed between the spring constants, such that $\alpha_1 = \alpha_2 + \alpha_K$. This relation was consistent with Lorman's concept of creep;[4] he cited seepage as the major cause of creep and the observed fact that recovery at a given time was a linear function of the previously applied stress indicating a constant value of α_K.

Thus, at low stresses at which little internal break-down would occur, α_2 would be very small and the completed seepage effect essentially the same as the completed delayed recovery. At high stresses more break-down would occur; α_2 would be larger which, in accordance with the assumed relation

between spring constants, would increase α_1, thereby increasing the effect of seepage upon creep.

Eq. (1) can thus be written in terms of only four constants. Hence

$$\dot{\epsilon}_c = C\,e^{-b}e^{-t/T_M} + \sigma\,\alpha_1(e^{-t/\tau_1} + 10e^{-10t/\tau_1}) \quad \text{.....(2a)}$$

$$\epsilon_c = Ce^{-b}T_M(1 - e^{-t/T_M}) + \sigma\,\alpha_1(2 - e^{-t/\tau_1} - e^{-10t/\tau_1}) \quad \text{.....(2b)}$$

Using the results of Series I and III, the Maxwell constants b and T_M were determined from the straight-line portion of the logarithm of "measured" creep rate versus time curves, and the Kelvin constants α_1 and τ_1 then evaluated from the creep-time curves. The Maxwell spring constants α_M

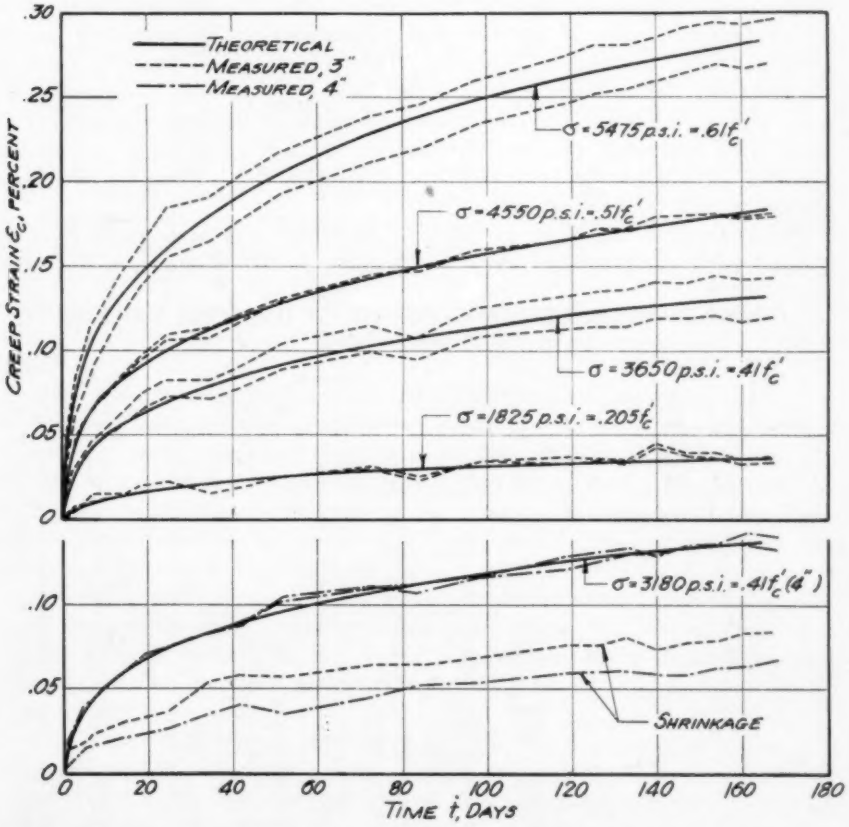

Fig. 9—Creep curves obtained with 3 x 10-in. cylinders of Mix 1:1 stressed respectively at 20.5, 41, and 61 percent of 28-day compressive strength, compared with curves of 4 x 10-in. cylinders stressed at 41 percent of compressive strength as well as with theoretical curves, Eq. (2b). Series III, test results from two cylinders in rig

were also determined for Series III by measuring instantaneous recovery at completion of the test. These constants were then used in Eq. (2) and the creep equations written for each mix and stress level of Series I and III.

The model constants are shown in Table 7, and the creep as obtained from Eq. (2) is plotted in Fig. 3, 4, and 5 for Series I, and Fig. 7, 8, and 9 for Series III, where they are compared with the actual test results.

Although Eq. (2) gives an excellent correlation between theoretical and measured creep, it gives no indication of the contributions of the four components to the total creep, as shown in Eq. (1). By determining α_K, and making use of the suggested relation between the spring constants and the retardation times, it is possible to evaluate these contributions.

The average values of the spring constants α_M and α_K were found from the results of Series II for the 4 in. diameter cylinders. By assuming that the ratio α_K/α_M as found for the 4 in. diameter cylinders was the same as those of 3-in. cylinders, values of α_K were found corresponding to values of α_M as given in Table 7 for the 3-in. cylinders. These constants are shown in Table 8.

Combining Tables 7 and 8, and letting $\alpha_2 = \alpha_1 - \alpha_K$ (but never less than zero) and $\tau_2 = \tau_K = \tau_1/10$, the model constants of the four components of creep were evaluated and the contribution of each component determined from Eq. (1) for $t = 28$ and $t = 250$ days to show the relative contributions after short and long periods of loading.

These constants are given in Table 9; contribution of each model element to total creep for the two times, $t = 28$ and $t = 250$, is shown in Table 10.

TABLE 7—MODEL CONSTANTS FOR EQ. (2) TEST SERIES I AND III, SUSTAINED LOADS

Test series	Mix	Diameter, in.	f_c', psi	Loading		Maxwell			Kelvin		
				σ, psi	Percent f_c'	T_M, days	b	$\alpha_M \times 10^6$ sq in. per lb	τ_1, days	$\alpha_1 \times 10^6$ sq in. per lb	$\varphi_1 \times 10^6$ sq in. per lb-day
I	1:6	4	5040	1590	31.5	271	12.87	—	6.8	0.078	1.15
				3180	63	395	12.02	—	19.9	0.195	0.98
					Average	333					1.07
	1:2.5	4	6160	1590	26	140	12.23	—	8.7	0.112	1.29
				3180	52	133	11.51	—	15.3	0.196	1.28
					Average	137					1.29
	1:1	4	9360	1590	17	291	12.97	—	20.8	0.114	0.55
				3180	34	216	12.17	—	22.5	0.146	0.65
					Average	254					0.60
III	1:6	3	6100	1245	20.5	59.0	11.73	0.198	2.9	0.040	1.38
				2480	40.5	59.5	11.19	0.278	15.9	0.099	0.62
				3720	61	71.2	10.87	0.264	22.1	0.175	0.79
					Average	63.2		0.246			0.93
	1:2.5	3	6490	1330	20.5	72.5	11.85	0.274	6.7	0.061	0.91
				2640	40.5	97.6	11.49	0.228	16.7	0.133	0.80
				3970	61	96.0	10.68	0.240	10.4	0.173	1.66
					Average	88.7		0.247			1.12
	1:1	3	8940	1825	20.5	64.0	12.21	0.245	3.4	0.021	0.62
				3650	41	116.2	11.70	0.220	18.7	0.081	0.43
				4550	51	141.2	11.59	0.216	19.6	0.101	0.52
				5475	61	124.2	11.10	0.211	18.0	0.132	0.73
					Average	111.4		0.223			0.58
	1:6	4	4840	3180	66	103.0	11.40	0.226	27.3	0.216	0.79
	1:2.5	4	5870	3180	54	88.0	10.99	0.226	13.6	0.111	0.82
	1:1	4	7750	3180	41	137.3	11.87	0.234	24.7	0.110	0.45

TABLE 8—INSTANTANEOUS AND DELAYED ELASTICITY SPRING CONSTANTS α_M AND α_K

Test series	Mix	Diameter, in.	f_c', psi	Loading		$\alpha_M \times 10^6$ sq in. per lb	$\alpha_K \times 10^6$ sq in. per lb
				σ, psi	percent f_c'		
	1:6	4	5110	1590	31	0.209	0.0462
				3180	62	0.208	0.0471
						Average 0.209	0.0467
II	1:2.5	4	5670	1590	28	0.247	0.0733
				3180	56	0.245	0.0645
						Average 0.246	0.0689
	1:1	4	9070	1590	17.5	0.226	0.0660
				3180	35	0.211	0.0519
						Average 0.219	0.0590
III	1:6	3	6100	—	—	Average 0.246	0.055*
	1:2.5	3	6490	—	—	Average 0.247	0.070*
	1:1	3	8940	—	—	Average 0.223	0.060*

*Based on ratio of α_K/α_M for 4 in. diameter cylinders.

The model constants of Table 9 were also used to compute the theoretical creep of 4 in. diameter cylinders due to periodically sustained loads, and the corresponding results compared graphically with the results of Series II as shown in Fig. 6. It was noted that although there was rather good agreement between the theoretical and measured creep during the loading-unloading periods, there was some deviation during the period of sustained loading after the third loading-unloading cycle. Because the creep due to the Kelvin elements after a long period of time reaches a constant value $\sigma\ (\alpha_1 + \alpha_2 + \alpha_K)$, it was assumed that most of the deviation was due to the constant T_M of the Maxwell element. The values of T_M were therefore adjusted to bring the theoretical creep into better agreement with the test results.

Although there is considerable variation between the adjusted values of T_M and those originally proposed for Series II, it is noted that the order of magnitude has remained unchanged, being 200 to 400 days for Mix 1:6, 100 to 300 days for Mix 1:2.5, and 200 to 300 days for Mix 1:1. The value of T_M appears to be unaffected by the size of the test specimen, as seen by comparing the results of tests on the 3 and 4 in. diameter cylinders of Series III, but greatly influenced by the type of cement used, as seen by comparing the values of T_M found for the 4 in. diameter cylinders of Series I (Saylor's cement) with those of the 4-in. cylinders of Series III (Lone Star cement).

The value of the constant b of the Maxwell element is stress-dependent, slightly affected by mix, and relatively uninfluenced by specimen size or cement quality.

The retardation time τ_1 of Kelvin Element (c) (the seepage element) is stress-dependent, which is consistent with the premise that it would take longer for a higher moisture gradient to decrease sufficiently for the seepage effect to cease. The fluidity φ_1 is essentially stress-independent. The combination of increasing τ_1 and constant φ_1 gives a stress-dependent α_1, which is consistent with the test results and the assumed importance of the seepage effect with increasing stress.

TABLE 9—MODEL CONSTANTS OF FOUR-ELEMENT MECHANICAL MODEL REPRESENTING CREEP AND CREEP-RECOVERY, EQ. (1)

Mix	Diameter, in.	Loading		Maxwell		Kelvin								
						Seepage			Breakdown			Recovery		
		σ, psi	Percent f_c'	T_M, days	b	$\alpha_1 \times 10^6$ sq in. per lb	τ_1 days	$\varphi_1 \times 10^6$ sq in. per lb-day	$\alpha_2 \times 10^6$	τ_2	$\varphi_2 \times 10^6$	$\dot{\alpha}_R \times 10^6$	τ_R	$\varphi_R \times 10^6$
1:6	4	1590	31.5	271	12.87	0.078	6.8	1.15	0.032	0.68	4.70	0.046	0.68	6.77
		3180	63	395	12.02	0.195	19.9	0.98	0.148	1.99	7.45	0.047	1.99	2.36
1:2.5	4	1590	26	140	12.28	0.112	8.7	1.29	0.039	0.87	4.49	0.073	0.87	8.39
		3180	52	133	11.51	0.196	15.3	1.28	0.131	1.53	8.56	0.065	1.53	4.25
1:1	4	1590	17	291	12.97	0.114	20.8	0.55	0.048	2.08	2.31	0.066	2.08	3.17
		3180	34	216	12.17	0.146	22.5	0.65	0.094	2.25	4.18	0.052	2.25	2.31
1:6	3	1245	20.5	59	11.73	0.040	2.9	1.38	0	0.29	0	0.055	0.29	19.0
		2480	40.5	60	11.19	0.099	15.9	0.62	0.044	1.59	2.77	0.055	1.59	3.46
		3720	61	71	10.87	0.175	22.1	0.79	0.120	2.21	5.42	0.055	2.21	2.49
1:2.5	3	1330	20.5	73	11.85	0.061	6.7	0.91	0	0.67	0	0.070	0.67	10.4
		2640	40.5	98	11.49	0.133	16.7	0.80	0.063	1.67	3.77	0.070	1.67	4.20
		3970	61	96	10.68	0.173	10.4	1.66	0.103	1.04	9.90	0.070	1.04	6.73
1:1	3	1825	20.5	64	12.21	0.021	3.4	0.62	0	0.34	0	0.060	0.34	17.7
		3650	41	116	11.70	0.081	18.7	0.43	0.021	1.87	1.12	0.060	1.87	3.21
		4550	51	141	11.59	0.101	19.6	0.52	0.041	1.96	2.09	0.060	1.96	3.06
		5475	61	124	11.10	0.132	18.0	0.73	0.072	1.80	4.00	0.060	1.80	3.34

As previously noted, the spring constant α_K for the Kelvin Element (b) (delayed elasticity element) is constant, resulting in a stress-dependent spring constant α_2 for the Kelvin Element (d) (local destruction or breakdown) in accordance with the assumed relation $\alpha_1 = \alpha_2 + \alpha_K$. The retardation times τ_2 and τ_K vary in the same manner as τ_1 because of the assumed relation $\tau_1 = 10\tau_2 = 10\tau_K$, but because of the similar order of magnitude of τ_2 and τ_K for all stress levels, it is reasonable to assume a constant value. Thus it is assumed that τ_K is stress-independent.

GENERALIZED CREEP EQUATIONS

Eq. (1) or (2) will represent the creep or creep-recovery for various mixes at various stress levels if the proper values of the model constants are known; the effective use of these equations requires a knowledge of the stress-dependent and mix-dependent constants as functions of the representative parameters. A single equation such as Eq. (1) can not represent the nonlinear response of the concrete at high stresses and the linear response at low stresses. Thus, it is necessary to introduce, for the range of low stresses, linear model equations with stress-independent parameters derived from the condition that when the creep at a given time t is plotted as a function of stress level, the slope of the nonlinear creep versus stress level curve be equal to the slope of the linear creep versus stress level curve at a stress ratio σ_o/f_c' at which significant deviation from linear response would become observable.

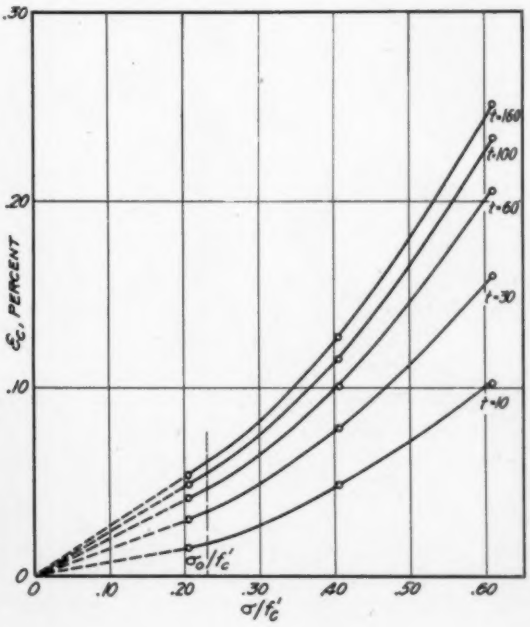

Fig. 10—Total creep after $t = 10$, 30, 60, 100, and 160 days as (interpolated) function of the applied specific stress ratio (σ/f_c') for 3 x 10-in. cylinders of Mix 1:6, Series III

TABLE 10—CONTRIBUTION OF VARIOUS MODEL ELEMENTS TO CREEP IN 10^{-5} IN. PER IN.

Test series		I (4-in. diameter)			
Mix	Period	28 days		250 days	
	Stress level	$0.315f_c'$	$0.63f_c'$	$0.315f_c'$	$0.63f_c'$
1:6	Element (a), ϵ_M	6.3	16.4	41.5	111.5
	Element (c), ϵ_1	12.2	46.9	12.4	62.0
	Element (d), ϵ_2	5.1	47.0	5.1	47.0
	Element (b), ϵ_K	7.3	15.0	7.3	15.0
	Total	30.9	125.3	66.3	235.5
	Stress level	$0.26f_c'$	$0.52f_c'$	$0.26f_c'$	$0.52f_c'$
1:2.5	ϵ_M	11.8	25.2	53.0	113.0
	ϵ_1	17.1	52.3	17.8	62.3
	ϵ_2	6.2	41.6	6.2	41.6
	ϵ_K	11.6	20.7	11.6	20.7
	Total	46.7	139.8	88.6	237.6
	Stress level	$0.17f_c'$	$0.34f_c'$	$0.17f_c'$	$0.34f_c'$
1:1	ϵ_M	6.2	13.5	38.0	76.2
	ϵ_1	13.5	33.0	18.1	46.4
	ϵ_2	7.6	29.9	7.6	29.9
	ϵ_K	10.5	16.5	10.5	16.5
	Total	37.8	92.9	74.2	169.0

Test series		III (3-in. diameter)					
Mix	Period	28 days			250 days		
	Stress level	$0.205f_c'$	$0.405f_c'$	$0.61f_c'$	$0.205f_c'$	$0.405f_c'$	$0.61f_c'$
1:6	Element (a), ϵ_M	17.8	30.8	44.0	46.5	81.4	131.0
	Element (c), ϵ_1	5.0	20.4	46.8	5.0	24.6	65.1
	Element (d), ϵ_2	0	10.9	44.7	0	10.9	44.7
	Element (b), ϵ_K	6.8	13.7	20.4	6.8	13.7	20.4
	Total	29.6	75.8	155.9	58.3	130.6	261.2
	Stress level	$0.205f_c'$	$0.405f_c'$	$0.61f_c'$	$0.205f_c'$	$0.405f_c'$	$0.61f_c'$
1:2.5	ϵ_M	16.6	25.0	55.8	50.2	92.4	204.0
	ϵ_1	8.0	28.5	63.5	8.1	35.1	67.7
	ϵ_2	0	16.6	40.9	0	16.6	40.9
	ϵ_K	9.3	18.5	27.8	9.3	18.5	27.8
	Total	33.9	88.6	188.0	67.6	162.6	340.4
	Stress level	$0.205f_c'$	$0.41f_c'$	$0.61f_c'$	$0.205f_c'$	$0.41f_c'$	$0.61f_c'$
1:1	ϵ_M	11.3	20.6	37.8	31.3	84.9	162.1
	ϵ_1	3.8	23.0	57.0	3.8	29.6	72.2
	ϵ_2	0	7.7	39.4	0	7.7	39.4
	ϵ_K	10.9	21.9	32.8	10.9	21.9	32.8
	Total	26.0	73.2	167.0	46.0	144.1	306.5

Although the "stress limits of linear response" σ_o were found to be time-dependent within the range of short times, they approached a time-insensitive value of roughly $0.20f_c'$ to $0.26f_c'$ after a few hundred days. It was therefore assumed that a limiting value of $\sigma_o/f_c' = 0.23$ could be used for all mixes, which is in agreement with the curves shown in Fig. 10-12. By assuming that the constants b and α_1 are constant for all values of $\sigma/f_c' \leqq \sigma_o/f_c'$, but vary linearly for $\sigma/f_c' \geqq \sigma_o/f_c'$, it was found that the stress-dependent constants can be evaluated from the following expressions:

$$b = \bar{b} \text{ when } \sigma/f_c' \leqq \sigma_o/f_c' \quad (3a)$$

$$b = \bar{b} - 2.62\,(\sigma/f_c' - \sigma_o/f_c') \text{ when } \sigma/f_c' \geqq \sigma_o/f_c' \quad (3b)$$

$$\alpha_1 = \bar{\alpha}_1 \text{ when } \sigma/f_c' \leqq \sigma_o/f_c' \quad (4a)$$

$$\alpha_1 = \bar{\alpha}_1 + 0.327 \times 10^{-6}\,(\sigma/f_c' - \sigma_o/f_c') \text{ when } \sigma/f_c' \geqq \sigma_o/f_c' \quad (4b)$$

where $\sigma_o/f_c' = 0.23$, and $\bar{b}$ and $\bar{\alpha}_1$ are mix and size-dependent.

If, as previously indicated, φ_1, α_K and τ_K are assumed to be stress-independent, the other stress-dependent constants can be found from

$$\tau_1 = \alpha_1/\varphi_1 \quad (5)$$

$$\alpha_2 = \alpha_1 - \alpha_K \geqq 0 \quad (6)$$

$$\tau_2 = \tau_1/10 \quad (7)$$

Using the results of Series I and III to evaluate $\bar{b}$ and $\bar{\alpha}_1$, and the results of Series I, II, and III to evaluate α_M, T_M, φ_1, α_K and τ_K, a set of generalized model constants was determined which can be used to predict the creep of

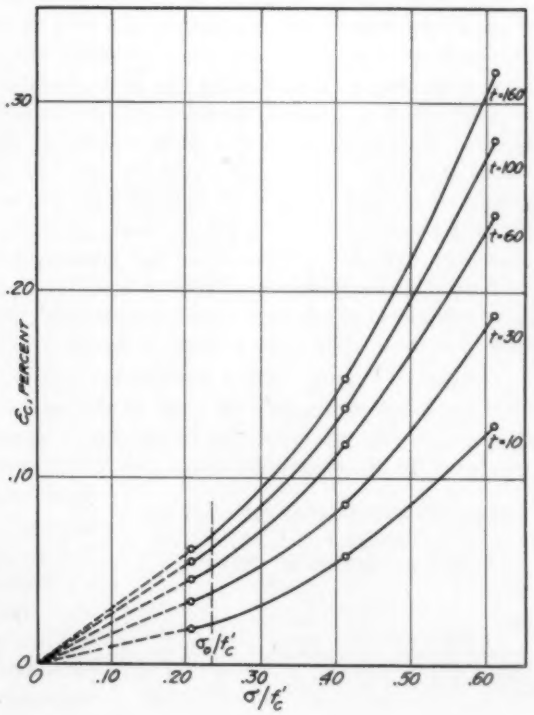

Fig. 11—Total creep after t = 10, 30, 60, 100, and 160 days as (interpolated) function of the applied specific stress ratio (σ/f_c') for 3 x 10-in. cylinders of Mix 1:2.5, Series III

TABLE 11—GENERALIZED MODEL CONSTANTS (LIMIT OF LINEAR RANGE $\sigma_o/f_c' = 0.23$)

Mix	Diameter, in.	Maxwell body				Kelvin bodies					
		$\alpha_M \times 10^9$, sq in. per lb	T_M, days	$\bar{b}$	$C_M \times 10^3$ 1/day	$\bar{\alpha}_1 \times 10^6$, sq in. per lb	$\bar{\tau}_1$ days	$\bar{\varphi}_1 \times 10^3$	$\alpha_K \times 10^6$	τ_K, days	$\varphi_K \times 10^3$
1:6	3	0.246	65	11.73	0.800	0.040	4	0.930	0.055	1	5
1:2.5	3	0.247	90	11.85	0.710	0.061	5	1.12	0.070	1	7
1:1	3	0.223	110	12.21	0.498	0.021	4	0.572	0.060	2	3
1:6	4	0.209	335	13.08	0.208	0.062	6	1.07	0.047	1	5
1:2.5	4	0.246	135	12.31	0.450	0.102	8	1.29	0.069	1	7
1:1	4	0.219	250	12.72	0.298	0.114	19	0.600	0.059	2	3

any mix subjected to any stress level. These constants are given in Table 11 and plotted as functions of the product of the volume concentration of the aggregate C_V (ratio of absolute volume of aggregates to absolute volume of concrete) and the water-cement ratio W_C (ratio of absolute volume of water to absolute volume of cement) (Fig. 14 and 15).

The Maxwell constants T_M and $\bar{b}$ for the 3 and 4 in. diameter cylinders are size-dependent with respect to both magnitude and variation with mix. Both T_M and $\bar{b}$ decrease essentially linearly with decreasing cement content for the 3-in. cylinders, but after decreasing initially, increase rapidly with the lean 1:6 mix for the 4-in. cylinders.

It is conceivable that this difference in behavior is a function of the free moisture content of the concrete at the time of application of the load. After the same curing period, the 3-in. cylinders are drier than the corresponding 4-in. cylinders, so that loading the 3-in. cylinder at an age of 28 days would be equivalent to loading the 4-in. cylinder at a later age, resulting in a lesser amount of limiting creep, or a lesser amount of time needed to reach a limiting value of creep. Thus, the "relative age" of the 3-in. cylinders would be greater than that of the 4-in. cylinders for the same curing conditions, resulting in a lower value of T_M for the 3-in. cylinders. It would thus be expected that 4-in. cylinders loaded at an age greater than 28 days would have Maxwell constants approaching those of the 3-in. cylinders loaded at 28 days, or 3-in. cylinders, which have been waterproofed to prevent moisture loss, would have constants approaching those of the 4-in. cylinders with no waterproofing.

The rate of "aging," which governs the rate at which the Maxwell constants decrease for any mix, is a function of the rate at which drying occurs. This drying rate depends upon the initial free moisture content which in turn depends on the water-cement ratio, and the ratio of water to aggregate. A low water-cement ratio and a low water-aggregate ratio result in little initial free moisture, most of the water being absorbed by the aggregate or combined with the cement; high water-cement and water-aggregate ratios result in considerable initial free moisture with consequent high drying rate. The

TABLE 12—VARIATION OF LIMIT OF LINEAR RANGE σ_o/f_c' WITH TIME FOR 3 IN. DIAMETER CYLINDERS

Mix	σ_o/f_c'				
	10 days	30 days	100 days	1000 days	∞
1:1	0.08	0.12	0.18	0.21	0.21
1:2.5	0.11	0.16	0.21	0.24	0.24
1:6	0.11	0.16	0.21	0.23	0.23

mixes used were such that Mix 1:6 had a high W/C ratio but a low W/A ratio, and Mix 1:1 a low W/C ratio but a high W/A ratio, so that the "aging" rate for these mixes is apparently higher than that of Mix 1:2.5 for which the decrease of the Maxwell constants was relatively small.

Equating the slopes at σ_o/f_c' of the linear and nonlinear creep versus stress-level curves plotted for time as a parameter, and letting b vary in accordance with Eq. (3a) or (3b), the creep equation, Eq. (1), can be written as a linear equation, for $\sigma/f_c' \leqq \sigma_o/f_c'$

$$\dot{\epsilon}_c = \frac{\sigma}{f_c'}\left[\frac{C_M e^{-t/T_M}}{\sigma_o/f_c'}\right] + \sum_{i=1,2,K} \frac{\sigma}{f_c'}\left[\alpha_i f_c' e^{-t/\tau_i}\right] \quad \text{.....(8a)}$$

$$\epsilon_c = \frac{\sigma}{f_c'}\left[\frac{C_M T_M (1 - e^{-t/T_M})}{\sigma_o/f_c'}\right] + \sum_{i=1,2,K} \frac{\sigma}{f_c'}\left[\alpha_i f_c' (1 - e^{-t/\tau_i})\right] \quad \text{.....(8b)}$$

with $C_M = Ce^{-\bar{b}}$, and T_M, α_i, τ_i constant; or a nonlinear equation, for $\sigma/f_c' \geqq \sigma_o/f_c'$

$$\dot{\epsilon}_c = C_M e^{2.62(\sigma/f_c' - \sigma_o/f_c')} e^{-t/T_M} + \sum_{i=1,2,K} \frac{\sigma}{f_c'}\left[\alpha_i f_c' e^{-t/\tau_i}\right] \quad \text{.....(9a)}$$

$$\epsilon_c = C_M T_M e^{2.62(\sigma/f_c' - \sigma_o/f_c')}(1 - e^{-t/T_M}) + \sum_{i=1,2,K} \frac{\sigma}{f_c'}\left[\alpha_i f_c'(1 - e^{-t/\tau_i})\right] \quad \text{.....(9b)}$$

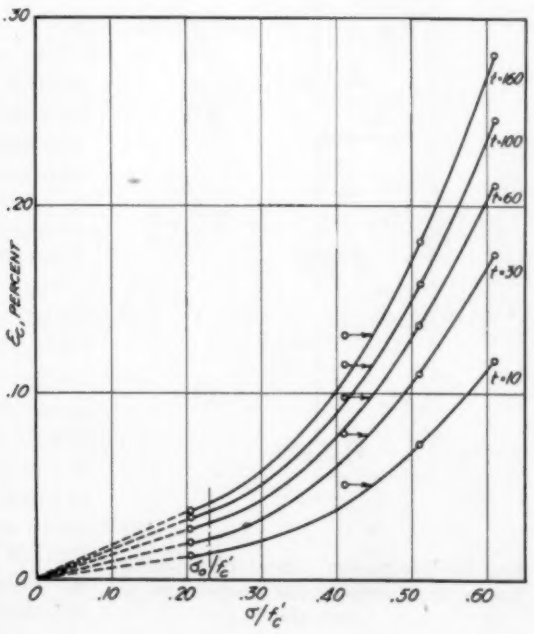

Fig. 12—Total creep after L = 10, 30, 60, 100, and 160 days as (interpolated) function of the applied specific stress ratio (σ/f_c') for 3 x 10-in. cylinders of Mix 1:1, Series III

for which α_1, α_2, τ_1, τ_2 are stress-dependent in accordance with Eq. (4) to (7).

The time-dependence of the limiting stress level σ_o/f_c' was found for the 3-in. diameter cylinders by equating the slope of the linear creep equation at various times t with that of the nonlinear equation, and solving for the corresponding value of σ_o/f_c'. The model constants used in this solution were those given in Table 11. Values as low as $\sigma_o/f_c' = 0.08$ were found for $t = 10$ days.

The variation of σ_o/f_c' as a function of time is shown in Table 12.

Predicted creep results

As a measure of the effectiveness of the proposed model and generalized creep equations (8) and (9), and the proposed values of the generalized model constants as given in Table 11 and Fig. 14 and 15, the expected creep was evaluated for four mixes of 3-in. diameter cylinders when subjected to four stress levels (Series IV).

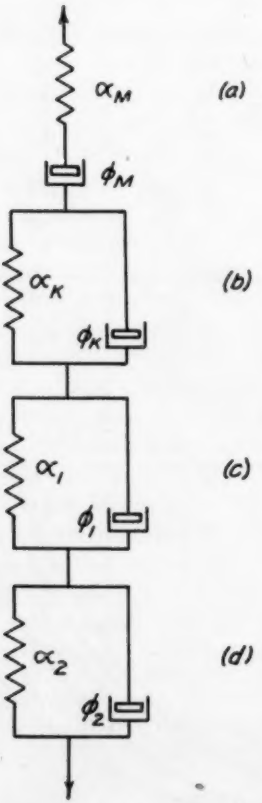

Fig. 13—Mechanical model representing the various deformation mechanisms of concrete

The generalized model constants for Mix 1:4 were obtained from Fig. 14 and 15. Using these constants, the creep equation constants were evaluated for each mix and stress level and used in the creep equations (8) and (9), and the predicted creep was computed.

The creep equation constants are given in Table 13 for the linear creep equation (8), and in Table 14 for the non-linear creep equation (9). The corresponding predicted creep curves for a 1:4 mix are plotted in Fig. 16 and compared with the actual results of Series IV, Mix 1:4.

There is good correlation between the predicted and the measured creep. For all levels of Mix 1:1 errors are negligible. This confirms the expectation that a rich concrete is essentially a viscoelastic material whose behavior can be described by such models as those proposed. The largest errors occur, as expected, in Mix 1:6 which is a partly viscoelastic and partly granular material whose behavior can not be completely described by visoelastic

TABLE 13—MODEL CONSTANTS FOR LINEAR CREEP EQ. (8), SERIES III, 3 x 10-IN. CYLINDERS

Mix	1:6	1:4	1:2.5	1:1
Ultimate strength f_c', psi	6000	6370	6900	8900
Stress { psi	1200	1270	1380	1780
Stress { percent f_c'	20	20	20	20
$\alpha_M \times 10^6$, in.$^2 \times$ lb^{-1}	0.246	0.247	0.247	0.223
$C_M \times 10^3$, day^{-1}	0.800	0.760	0.710	0.498
T_M, days	65	77	90	110
$\alpha_1 \times 10^6$, in.$^2 \times$ lb^{-1}	0.040	0.054	0.061	0.021
$\alpha_2 \times 10^6$, in.$^2 \times$ lb^{-1}	0	0	0	0
$\alpha_K \times 10^6$, in.$^2 \times$ lb^{-1}	0.055	0.064	0.070	0.060
τ_1, days	4	5	5	4
τ_2, days	—	—	—	—
τ_K, days	1	1	1	2
$\varphi_1 \times 10^3$, in.$^2 \times$ lb$^{-1} \times$ day^{-1}	0.930	1.05	1.12	0.572
$\varphi_2 \times 10^3$, in.$^2 \times$ lb$^{-1} \times$ day^{-1}	0	0	0	0
$\varphi_K \times 10^3$, in.$^2 \times$ lb$^{-1} \times$ day^{-1}	5.00	6.00	7.00	3.00

TABLE 14—MODEL CONSTANTS FOR NONLINEAR CREEP EQ. (9), SERIES III, 3 x 10-IN. CYLINDERS*

Mix	1:6			1:4			1:2.5			1:1		
f_c', psi	6000			6370			6900			8900		
Stress, psi	2110	3000	3900	2220	3180	4130	2420	3450	4480	3110	4450	5650
percent f_c'	35	50	65	35	50	65	35	50	65	35	50	64
$\alpha_M \times 10^6$	0.246	0.246	0.246	0.247	0.247	0.247	0.247	0.247	0.247	0.223	0.223	0.223
$C_M \times 10^3$	0.800	0.800	0.800	0.760	0.760	0.760	0.710	0.710	0.710	0.498	0.498	0.498
T_M	65	65	65	77	77	77	90	90	90	100	100	110
$\alpha_1 \times 10^6$	0.079	0.128	0.173	0.093	0.142	0.191	0.100	0.149	0.204	0.060	0.109	0.158
$\alpha_2 \times 10^6$	0.024	0.073	0.118	0.029	0.078	0.127	0.030	0.079	0.134	0	0.049	0.098
$\alpha_K \times 10^6$	0.055	0.055	0.055	0.064	0.064	0.064	0.070	0.070	0.070	0.060	0.060	0.060
τ_1	9	14	19	9	14	18	9	13	18	10	19	28
τ_2	1	1½	2	1	1½	2	1	1½	2	—	2	3
τ_K	1	1	1	1	1	1	1	1	1	2	2	2
$\varphi_1 \times 10^3$	0.930	0.930	0.930	1.05	1.05	1.05	1.12	1.12	1.12	0.572	0.572	0.572
$\varphi_2 \times 10^3$	2.70	5.30	6.35	3.29	5.78	6.98	3.38	6.13	7.36	0	2.56	3.55
$\varphi_K \times 10^3$	5.00	5.00	5.00	6.00	6.00	6.00	7.00	7.00	7.00	3.00	3.00	3.00

*Units are same as for Table 13.

TABLE 15—RELAXATION OF SYMMETRICALLY PRESTRESSED MEMBER DUE TO CREEP ALONE, AND DUE TO CREEP AND SHRINKAGE

Mix 1:6 3 in. diameter cylinder $f_c' = 6000$ psi

Ratio of stable steel stress to initial steel stress	$\sigma_i/f_c' = 0.20$		$\sigma_i/f_c' = 0.40$	
	$f_{si} = 30{,}000$ psi	$f_{si} = 120{,}000$ psi	$f_{si} = 30{,}000$ psi	$f_{si} = 120{,}000$ psi
	$p^* = 0.04$	$p = 0.01$	$p = 0.08$	$p = 0.02$
Creep alone	0.710	0.903	0.600	0.845
Creep and shrinkage	0.409	0.814	0.354	0.760

*$p = A_s/A_c$.

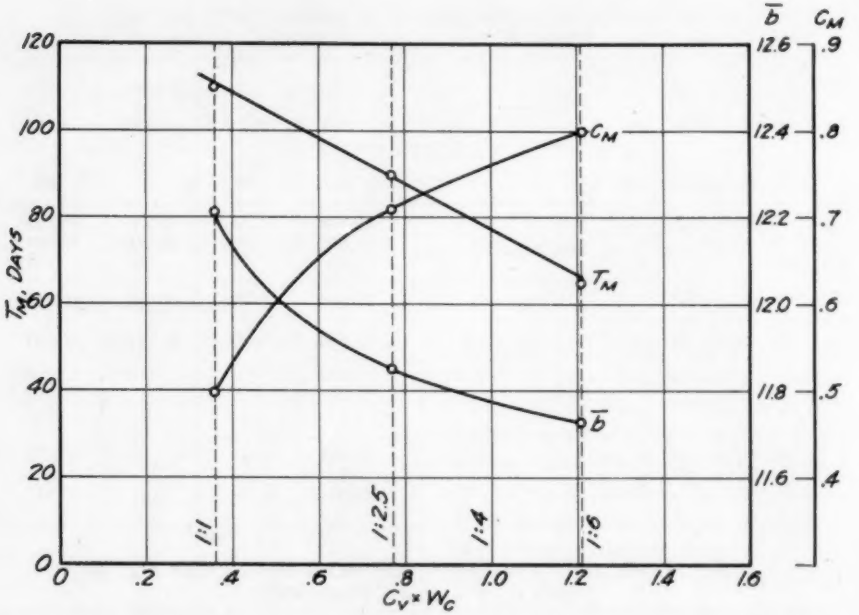

Fig. 14—Variation of parameters of Maxwell element of model with mix (3 x 10-in. cylinders)

models alone. The error for all stress levels is approximately 17 percent which is not unusually high in view of the variation of even such simple properties as compressive strength or modulus of rupture of ordinary concrete mixes. The errors for Mix 1:2.5 range from 5 to 14 percent, and for Mix 1:4, from 1½ to 12½ percent. Note that from the variation of the generalized model constants as given in Fig. 14 and 15, the creep equations (8) and (9), and the compressive strength of test cylinders, it was possible to predict closely the creep of a mix such as Mix 1:4 which had not been tested before.

Applications of generalized creep equations

A number of simplifying assumptions can be introduced by considering that in most practical problems, either the response of certain of the model elements is simplified because the time scale of the imposed load or enforced deformation is much shorter or much longer than the times τ or T_M governing the various model responses, or the variation with time of the applied loads or enforced deformations is either rapid enough to disregard the time dependence of the parameters or slow enough to disregard the effect of this variation.

To illustrate the use of the proposed model, a one-dimensional problem is discussed in which the time scale is such that the Kelvin elements can be disregarded (i.e. the time t is large compared with τ_1, τ_2 and τ_K) so that the

response is due primarily to the Maxwell element for which the fluidity is only time-dependent in the linear range, but time and stress-dependent in the nonlinear range.

A problem in which only the Maxwell element can be assumed to be effective is the long-time loss of prestressing in a symmetrically prestressed member.

Due to application of initial prestress, the concrete will creep, thereby allowing the steel to contract reducing its tensile force as well as the compressive force in the concrete which, in turn, reduces the subsequent creep. Equality of strain rates in steel and concrete of the self-equilibrating system is expressed by

$$-\frac{\dot{P}\alpha_s}{A_s} = \frac{\dot{P}\alpha_M}{A_c} + \frac{P}{A_c}\varphi_M \qquad \text{(a)}$$

where $\alpha_s = 1/E_s$ is the spring constant of the steel, A_s and A_c are the areas of steel and concrete respectively, and $P(t)$ the prestressing force.

If the initial force P_i causes a concrete stress $\sigma_i < \sigma_o$ the fluidity varies in accordance with the first term of Eq. (8a). The solution of equation (a) is therefore

$$P = P_i\, e^{k_1(e^{-t/T_M} - 1)} \qquad \text{(b)}$$

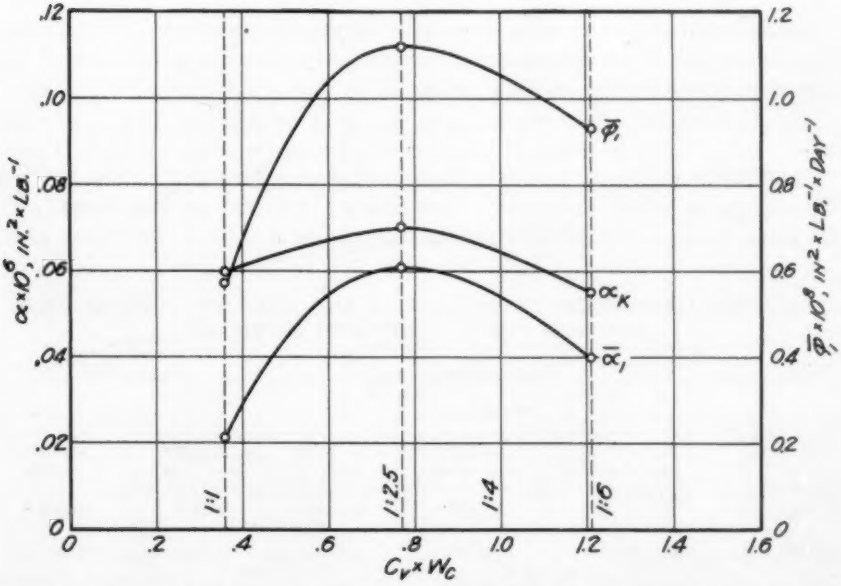

Fig. 15—Variation of parameters of Kelvin elements of model with mix (3 x 10-in. cylinders)

where

$$k_1 = \frac{C_M T_M}{\sigma_o (\alpha_M + \alpha_s A_c/A_s)}$$

If the initial concrete stress $\sigma_i > \sigma_o$, the fluidity is given by the first term of Eq. (9a). Thus

$$e^{-k_2 P} = e^{-k_2 P_i} + k_2(1 - e^{-t/T_M}) \quad \text{.....(c)}$$

where

$$k_2 = \frac{2.62}{f_c' A_c}$$

and

$$k_3 = \frac{2.62\, C_M T_M\, e^{-2.62\, \sigma_o/f_c'}}{f_c' A_c \left(\frac{\alpha_M}{A_c} + \frac{\alpha_s}{A_s}\right)}$$

Eq. (c) is valid until time $t = t_o$ at which $\sigma = \sigma_o$. For $t \geqq t_o$, the fluidity is again assumed to be stress-independent, and the relation expressed by

$$P = P_i\, e^{k_1(e^{-t/T_M} - e^{-t_o/T_M})} \quad \text{.....(d)}$$

From Eq. (b) and (d), with given areas of steel and concrete, it is possible to evaluate the stress relaxation in the steel and concrete, and therefore the loss of prestress in the nonlinear material.

To illustrate the stress relaxation in the steel or concrete, four conditions were studied for a concrete of Mix 1:6 whose constants are given in Tables 13 and 14, considering a steel of modulus of elasticity $E_s = 29 \times 10^6$ psi. Two levels of initial prestress f_{si} were selected (120,000 psi and 30,000 psi), for each of which two initial concrete stress levels σ_i/f_c' were used (20 and 40

TABLE 16—COMPARISON OF THEORETICAL AND MEASURED LOSS OF STEEL PRESTRESS DUE TO CREEP AND SHRINKAGE

Steel stress, psi	Staley and Peabody			Theoretical	
	Mix 1:5			Mix 1:6	
	$\sigma_i = 0.19 f_c'$ $p^* = 0.0276$	$\sigma_i = 0.31 f_c'$ $p = 0.049$	$\sigma_i = 0.49 f_c'$ $p = 0.0735$	$\sigma_i = 0.20 f_c'$ $p = 0.04$	$\sigma_i = 0.40 f_c'$ $p = 0.08$
Initial	32,000	31,000	31,800	30,000	30,000
At 380 days	10,460	8260	9020	12,340	10,630
Percent of initial	32.7	25.6	28.3	41.1	35.4

*$p = A_s/A_c$.

percent) with a concrete area of 100 sq in. Applying Eq. (b) and (d), the ratios of stable steel stresses to initial stresses were computed (Table 15).

Eq. (b) and (d) will give the relaxation of prestressing due to the creep of the concrete; however, it should be remembered that further loss of prestressing is caused by shrinkage unless the prestressing is done after shrinkage is completed. The decrease in steel stress due to a shrinkage ϵ_s in the concrete is given by

$$f_s = \epsilon_s E_s \frac{1}{1 + np} \qquad \text{(e)}$$

Using a shrinkage of 0.0004 in. per in. for Mix 1:6 prestressed at 28 days in accordance with the shrinkage curve shown in Fig. 15, the loss in steel stress was evaluated for the four combinations of initial steel and concrete stresses described above. This loss of prestress was combined with the loss due to creep and the remaining stable stress evaluated as shown in Table 15.

In tests conducted by Staley and Peabody[20] the remaining steel stresses at 380 days were observed for a concrete of Mix 1:5 which had been prestressed to approximately 20, 30, and 50 percent of its ultimate strength by means of

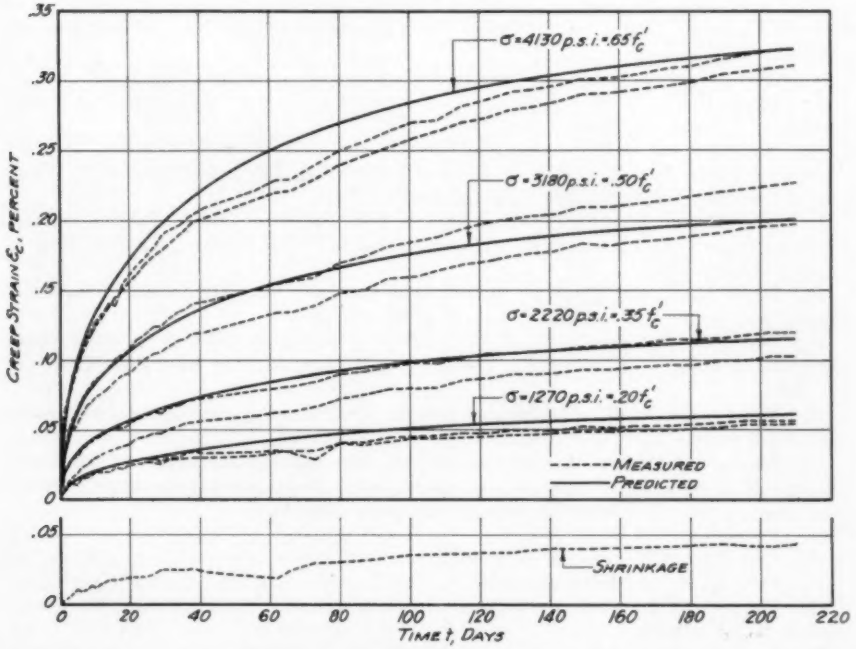

Fig. 16—Comparison of creep curves obtained with 3 x 10-in. cylinders of Mix 1:4 stressed respectively at 20, 35, 50, and 65 percent of 28-day compressive strength, with theoretically predicted curves Eq. (8) and (9) using parameters established in tests of different mixes, Series IV

an initial steel tensile prestress of approximately 32,600 psi. The decreasing steel and concrete stresses were due to shrinkage and creep. The theoretical relaxation due to shrinkage and creep at 380 days was evaluated for Mix 1:6 and compared with the results of Staley and Peabody (Table 16). Agreement between the theoretical and measured results is satisfactory.

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CONCRETE BRIEFS

Notes from Field and Office

Effect of Column Width on Continuous Beam Moment 1143
Permissible Concrete Shear Stresses of the 1957 British Code of Practice 1146

Effect of Column Width on Continuous Beam Moment

By HENRI C. HEPP and THOR GERMUNDSSON

By HENRI C. HEPP*

The following comments on an analysis in the Portland Cement Association bulletin "Continuity in Concrete Building Frames" may be of interest to ACI JOURNAL readers.

The PCA analysis proposes that the theoretical moment curve (computed assuming concentrated reactions) should be raised a value $1/6\,Va$ or, in other words, the theoretical moment at the face of column should be increased by $1/6\,Va$ and the moment at midspan should be reduced by $1/6\,Va$.

The following analysis shows that the theoretical moments are practically unmodified along the span and reduced only over the width of the column.

Following the PCA assumption (not discussed here) on the distribution of the shear over the width of the column, the area of the modified shear diagram (shaded area) between axis and face of column is correctly given by $1/3\,Va$ (Fig. 1). However, the shape of the moment diagram will consequently be modified over the width of the column. As the theoretical moment at the axis of the column decreases to a value M_d, the reduction $1/3\,Va$ must be applied between points d and b and not c and b. The value of the moment at b remains practically unchanged as shown by the following investigation.

The assumption of the value $1/3\,Va$ for the shaded area in the shear diagram corresponds to a parabola of the second order and implies a triangular distribution of pressure over the width of the column. If the beam is loaded with the concentrated load V and the column reactions take the assumed triangular distribution indicated in Fig. 2(a), then the superposition of this corrective loading on the theoretical loading will give the final modified moment diagram. To find the moment M' due to the corrective loading, Fig. 2(a), we compute first the end rotation α_o in the simply supported beam of Fig. 2(b) where I is constant:

$$\alpha_o = \frac{1}{EI}\left(\frac{Va}{6}\cdot\frac{L}{2} - \frac{Va}{6}\cdot\frac{a}{8}\right)$$

For equal end moments $M = 1$, we have $\alpha_1 = L/2EI$. Since the tangent is assumed horizontal at column axis, it follows that $\alpha_o - M'\,\alpha_1 = 0$, and

$$M' = \frac{\alpha_o}{\alpha_1} = \frac{Va}{6} - \frac{Va^2}{24L}$$

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*Civil Engineer, Ammann and Whitney, New York, N. Y.

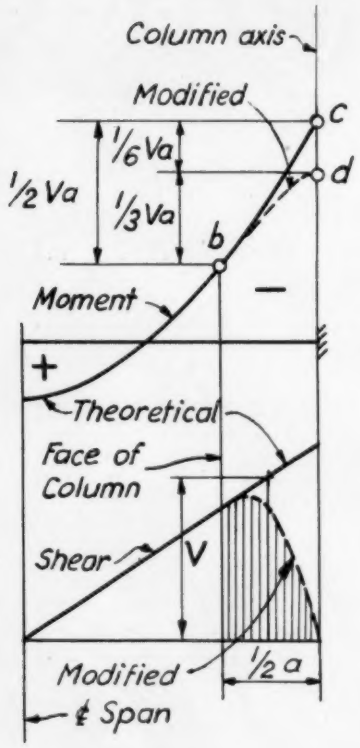


Fig. 1

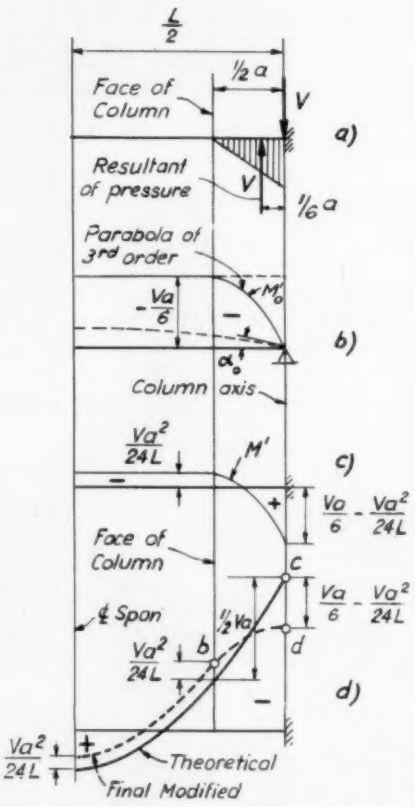


Fig. 2

at column axis. Fig. 2(c) represents the moment diagram for the corrective loading. Superposition of moment M' with the theoretical moment shall give the final modified moments, Fig. 2(d).

The value $Va^2/24L$ is very small and may be neglected for practical cases. For instance, with $a = L/10$, $V = wL/2$

$$M_b = \frac{wL^2}{12}$$

$$\frac{Va^2}{24L} = \frac{wL^2}{4800}$$

or 0.25 percent of M_b. (The method proposed in the PCA analysis would increase M_b by $Va/6 = wL^2/120$ or 10 percent of M_b and decrease $+M$ by 20 percent.)

Neglecting the value $Va^2/24L$, the moments at face of column and along the span remain unchanged from the theoretical moments and may be used in the design of beams and girders; the moment at face of column being equal to $M_c - 1/2\,Va$. For frames, the moment M_d (equal to $M_c - 1/6\,Va$) at column axis is rarely critical because of the increased effective depth of the beam over the column. This increase may be taken as a fictitious haunch with a slope approximately equal to 1 on 3 on the bottom of the beam.

By THOR GERMUNDSSON*

Mr. Hepp's proposal that beam moments at column face and midspan in a continuous beam remain the same as theoretical moments computed from a center line diagram is based on assumptions regarding the reactions on the beam within the column.

There is another factor to consider. A column contributes additional stiffness or increase in moment of inertia to the ends of the member, similar to a haunch, that requires correction to midspan and column face moments. The correction is approximately $VLa/6$ as indicated in "Continuity in Concrete Building Frames."

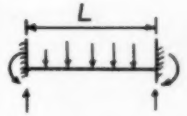
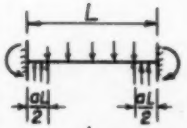
Table 1 illustrates the effect of various assumptions relative to bearing distribution. to theoretical moments at column face indicated in Table 1 are $VLa^2/12$ and $VLa^2/4$ for Assumptions (2) and (3) respectively. The midspan moment corrections are the same as those given in Table 1 for the column face. For usual small values of a no correction is needed to the theoretical moment curve at either the midspan or the column face.

As shown in Table 2 the effect of increase in moment of inertia at the column has a more significant meaning. Moment corrections were determined using the PCA publication "Handbook of Frame Constants," Table 56, p. 23 for different values of a.

The column face moment correction is the same as shown in Table 2 for midspan mo-

TABLE 1

MOMENT AT FACE OF COLUMN

Assumed reaction	Theoretical moment, ℄ column	Moment column face	Column face, moment correction	
(1) Theoretical (Reaction conc. at col.℄)	$VL/6$	$VL/6-\left(\frac{VLa}{2}-\frac{VLa^2}{4}\right)$	0	
(2) Reaction uniform from col. face to col. ℄	$VL/6$	$VL/6-\left(\frac{VLa}{2}-\frac{VLa^2}{3}\right)$	$\frac{VLa^2}{12}$	
(3) Reaction concentrated at face of col.	$VL/6$	$VL/6-\left(\frac{VLa}{2}-\frac{VLa^2}{2}\right)$	$\frac{VLa^2}{4}$	

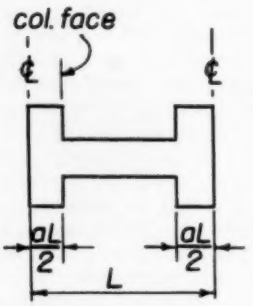
In the first line, theoretical moments are computed with the reaction concentrated at center line of column. In the second line, reaction is assumed distributed uniformly from face to center line of column. Reaction in the third line is assumed concentrated at column face. In a structure, the reaction probably varies from a maximum at column face to a minimum at column center line. Corrections ment correction. For usual values of a the correction raises the moment curve at midspan and column face by $VLa/6$ as indicated in "Continuity in Concrete Building Frames."

For the above reasons it is believed that the correction $VLa/6$ is reasonable. It results in less reinforcing steel in the positive moment region, and more in the negative moment region. This may increase ultimate

*Member American Concrete Institute, Staff Adviser, Portland Cement Association, Chicago, Ill.

TABLE 2

a	Correction to midspan moment
0.1	-0.16 VLa
0.2	-0.15 VLa
0.3	-0.14 VLa



moment capacity of the column face cross section with a corresponding decrease at midspan. In effect the critical section at ultimate strength may be shifted from column face to midspan. For this reason the correction may be more academic than real.

"Continuity in Concrete Building Frames" is currently being revised. The correction of $VLa/6$ to be applied at the column face and at midspan will remain, but the text will be changed to include some of the discussion presented here.

Permissible Concrete Shear Stresses of the 1957 British Code of Practice

By C. B. WILBY*

The writer has now tested 83 shear failures of beams and from this experience feels bound to adversely criticize the new 1957 British Code of Practice with regard to the permissible shear stresses specified for concrete. This code allows the permissible shear stresses to equal 40 psi plus 6 percent of the allowable compressive stress in flexure. The allowable bending stress is one-third of the strength u of the "works" cube according to this code of practice. Therefore we can derive the equation:

$$s = 40 + 0.06\frac{u}{3}$$

$$= 40 + 0.02\,u \quad \ldots\ldots\ldots(1)$$

The results of 38 beams with plain webs tested to destruction in shear are recorded in Table 1. This table gives the ultimate strength u of the concrete of each beam, and the ultimate shear stress of each beam v. From u, the value of the permissible shear stress s is computed using Eq. (1) and Table 1 also records the resulting factor of safety, $FS = v/s$.

Many engineers will feel that the factor of safety against the failure of a plain webbed beam in shear should be greater than the factor of safety against failure in bending. This latter is not less than 2 (in the case of a flexural tension failure) for normal design in accordance with the British code under discussion. Probably most authorities in Britain would desire a factor of safety against failure in shear of at least 2.2. The factors of safety in Table 1, on this basis, are most alarming; on only ten occasions out of 38 tests is the factor of safety greater than or equal to 2.2. Of the 28 tests with an inadequate factor of safety, many are very dis-

*Member American Concrete Institute, Structural Engineer, Cheadle Hulme, Cheshire, England.

TABLE 1—TEST RESULTS OF 38 BEAMS WITH PLAIN WEBS TESTED TO DESTRUCTION IN SHEAR

Beam No.	u	v	s	FS	t	R
1(a)	4690	355	134	2.65	351	2.12
1(b)	4690	332	134	2.48	351	3.18
2(a)	5290	338	143	2.36	315	2.12
2(b)	5290	314	143	2.20	315	3.18
3(a)	5010	224	140	1.60	393	2.14
3(b)	5010	214	140	1.53	393	3.22
4(a)	5380	264	148	1.78	330	2.13
4(b)	5380	211	148	1.43	330	3.20
5(a)	5820	231	146	1.58	310	2.12
5(b)	5820	214	146	1.46	310	3.18
6(a)	4170	358	123	2.91	282	2.09
6(b)	4170	230	123	1.87	282	3.14
8(a)	9770	625	236	2.64	358	2.14
8(b)	9770	417	236	1.76	358	3.21
9(a)	9400	618	228	2.71	276	2.14
9(b)	9400	189	228	0.828	276	3.21
10(a)	8680	515	214	2.41	397	2.14
10(b)	8680	254	214	1.185	397	3.21
11(a)	9240	262	225	1.165	425	2.14
11(b)	9240	240	225	1.07	425	3.21
12(a)	9240	435	225	1.93	390	2.08
12(b)	9240	267	225	1.19	390	6.23
13(a)	9010	795	220	3.61	402	2.09
13(b)	9010	263	220	1.20	402	6.28
14(a)	8960	324	219	1.48	405	2.08
14(b)	8960	290	219	1.32	405	6.23
15(a)	8820	449	216	2.07	425	2.10
15(b)	8820	263	216	1.22	425	6.31
16(a)	8750	654	175	3.74	387	2.08
16(b)	8750	283	175	1.62	387	6.25
17(a)	9100	258	222	1.16	370	4.14
18(a)	9770	305	236	1.295	358	3.45
18(b)	9770	236	236	1.0	358	4.83
19(a)	9400	448	228	1.96	276	2.79
19(b)	9400	267	228	1.17	276	5.57
20(a)	8680	438	214	2.04	397	2.07
20(b)	8680	249	214	1.16	397	6.21
D, D1	5000	240	140	1.715		33.8

turbing; 12 tests recorded a factor of safety of even less than 1.3, and the lowest factor of safety was 0.828.

From Eq. (1), if $s = 152$ psi, then $u = 5600$ psi. Twenty-seven of the tests concern concrete with a compressive strength in excess of 5600 psi. Such beams would therefore qualify to be designed with $s = 152$ psi according to the code of practice. Adopting a factor of safety of 2.2, v should not therefore be less than 334 psi. In these 27 tests, the ultimate shear stress was less than 334 on 17 occasions!

If $s = 130$, then from Eq. (1), $u = 4500$. Nine of the remaining 11 tests could therefore be designed upon such a basis according to the code. Using a factor of safety of 2.2, v should not be less than 286 psi. Of these nine tests, the ultimate shear stress was less than 286 on five occasions!

Even if one never designed with a permissible shear stress in excess of 130 psi, then out of the above 36 tests, the ultimate shear stress is less than 286 on 19 occasions. It can therefore be dangerous to use permissible concrete shear stresses as specified by the 1957 British Standard Code of Practice. The code only considers the crushing strength of the concrete, Eq. (1). The writer believes that a shear failure is akin to the tensile strength of the concrete,[1,2,3] and that the tensile strength (as measured by test briquets) is not directly proportional to the compressive strength of concrete. This is where the code errs in dealing with high strength concretes.

Table 1 records the tensile briquet strengths t of the beams in question. A comparison of the values of u with the corresponding values of t would indicate that t was independent of u for the tests under consideration. This is of course contrary to the views of many investigators and to the ideas of the British code of practice.

Also, the strength of the concrete by no means has the prime influence upon the shearing strength of a beam; other factors[1,2] have a greater aggregate effect upon the shearing strength. Shear strength is less for beams with longer shear spans/effective depth ratios; the code has no doubt been biased by the majority of past research which

has chiefly dealt with ratios between 1 and 2. Beams in practice have much higher ratios R than this, and the results in Table 1 concern much higher values than used by previous investigators, to the writer's knowledge. The present tendency is for spans to become longer and shallower, and this is encouraged by the increased permissible stresses of the code. A reduction in the percentage of longitudinal tension reinforcement reduces the shearing strength of a beam. The percentage of such steel is often reduced these days when designers utilize high tensile deformed bars. Large amounts of longitudinal compression reinforcement reduce the shearing strengths of rectangular beams,[1,2] and such large quantities are often experienced at the supports of continuous beams. Modern practice tends if anything to increase the use of such reinforcement. Thus there is a general inclination for the shear strength of beams to be decreased by modern practice. Consequently a revision of ideas is necessary with regard to the safety of beams in shear.

As a result of tests and a study of this problem, the writer would propose that, irrespective of the concrete's crushing strength (providing concrete is of sufficient strength to be considered by the code of practice), permissible shear stress at working loads of a plain webbed beam should be limited to 106 psi, except in the case of beams of rectangular cross section with more than 0.17 percent of longitudinal compression reinforcement, when the permissible shear stress should then be limited to 79 psi. This latter recommendation would of course usually concern continuous T-beams.

The writer would then propose that shear reinforcement be designed only to withstand the shear in excess of these values.

The writer considers the 1957 British Code of Practice inadequate and even dangerous in the case of the higher strength concretes for the design of plain webbed beams in shear. When shear reinforcement is required and is designed by this code of practice, such designs are generally excessively conservative. The code for example never utilizes percentages of vertical stirrups of less than 0.5 percent. This is indeed a fallacy as smaller percentages of web reinforcement are most definitely effective for resisting shear (see Fig. 4 in Reference 2). The least web reinforcement which the writer has tested and found effective in shear is 0.079 percent.

The writer considers the above proposals conservative and always safe. They would remedy the present illogical state of affairs recommended by the code in which a beam either needs no shear steel at all or an excessively large quantity of such reinforcement, with detailing of these large amounts of steel often most troublesome.

The results quoted were obtained from tests made by the writer at Leeds University under the direction of Prof. R. H. Evans, with the exception of the beams No. D, D1. Other beams not recorded here were tested by the writer in his capacity as development engineer of Stuart's Granolithic Co., Ltd.

REFERENCES

1. Wilby, C. B., Ph.D. Thesis, "The Strength of Reinforced Beams in Shear," The University of Leeds, May, 1949.

2. Wilby, C. B., "The Strength of Reinforced Concrete Beams in Shear," *Magazine of Concrete Research* (London), No. 7, Aug., 1951.

3. Ockleston, A. J., "Load Tests on a Three Storey Reinforced Concrete Building in Johannesburg," *The Structural Engineer* (London), Oct. 1955.

APPENDIX

For the benefit of American readers it can be assumed that the crushing strength of a standard concrete cylinder $f_c' = 0.85\,u$.

From the 1956 ACI Building Code, the equivalent to Eq. (1) is:

$$s = 0.03\,f_c' = 0.03 \times 0.85\,u \quad \ldots\ldots\ldots (2)$$

with a stipulation that s must not exceed 90 psi. When this value of s is substituted in Eq. (2), $u = 3530$ psi. The tests recorded in Table 1 therefore all refer to concretes where the ACI code would limit s to 90 psi. For a factor of safety FS of 2.2, the ultimate shear stress v should therefore be 198 psi. Out of the 38 tests quoted only Beam 9 (b) does not meet with this requirement. The FS in this particular instance would be 2.1.

The 1956 ACI Building Code is therefore not embarrassed by results cited in Table 1.

CURRENT REVIEWS

of Significant Contributions in Foreign and Domestic Publications

Bridges

Competitive designs of the bridge over the Drava river at Ptuj (in Slovene)

J. STARIC, *Gradbeni Vestnik* (Ljubljana), V. 8, No. 47-50, 1956-57, pp. 109-122

Reviewed by J. J. POLIVKA

Ten competitive designs were submitted to the Road Administration in Slovenia, four of them in reinforced concrete, three in prestressed concrete, and three in steel. The winner of the competition was Krešimir Savor of Zagreb proposing a bridge in reinforced concrete, 46 ft wide (three lanes of 11 ft with two sidewalks), total length 640 ft. Bridge consists of two end spans at 115 ft and three spans at 137 ft, all spans continuous with two cantilever hinges in the center span; 8-in. slab is monolithically connected with four girders, 12 in. wide, with total depth increasing from approximately 5 to 10 ft toward the piers, where the girders have two bottom slabs 10 in. thick to provide compression zone, thus forming two parallel box girders.

Construction

Industrial floors (in Dutch)

L. F. DAMMERS, *Cement* (Amsterdam), V. 9, No. 5-6, June 1957, pp. 193-202

Reviewed by JOHN W. T. VAN ERP

In general two methods for constructing industrial floors are given: (1) monolithic floors in which the wearing course is formed by imbedding the wear-resistant material in the concrete before it hardens; (2) concrete floors with a separate wearing course.

Both methods are dealt with at length; mix proportions are given; finishing methods covered in detail include coloring and special chemically resistant floors, finishing machines, etc. The general conclusion is that the monolithic floor is best, but also the most expensive, as the wearing course has to be applied a couple of hours after the floor is cast, and has to be finished completely which usually means overtime work.

Forms and scaffolds for shell construction (Schalung und Rüstung—Moderne Schaltechnik)

BOEHM and LABUTIN, Wilhelm Ernst and Sohn, Berlin, 1957, 4th edition, 211 pp., 36 DM

SCHWEIZERISCHE BAUZEITUNG V. 75, No. 48 (Strässler)

Indicates methods of analysis and design, materials, and methods of construction for formwork especially for wide span shell construction.

Design

Determination of temperature stresses in reinforced concrete beams with variable sections (in Rumanian)

V. PETCU, *Industria Constructiilor si a Materialelor de Constructii* (Bucharest), V. 8, No. 11, Nov. 1957, pp. 636-643

Reviewed by J. J. POLIVKA

In most practical cases haunch beams are exposed to different temperature on the top and the bottom, especially in roof and floor structures. These thermal conditions can seriously affect the strength of the beams. Following cases are investigated: (1) Fully restrained beam with haunch at one end; (2) Similar unsymmetrical beam, one end simply supported; (3) Symmetrical fixed beam with two identical haunches. Practical examples (two-bay symmetrical frame, hinged beams rigidly connected with hinged columns of variable section) demonstrate the procedure of the structural analysis presented in

A part of copyrighted JOURNAL OF THE AMERICAN CONCRETE INSTITUTE, V. 29, No. 12, June 1958, *Proceedings* V. 54. Address P.O. Box 4754, Redford Station, Detroit 19, Mich. Where the English title only is given in a review, the book or article reviewed is in English. If it is followed by a foreign title the work reviewed is in that language. In those cases where the foreign title cannot conveniently be set in type or is not available, the language of the original article is indicated in parentheses following the English title. Copies of articles or books reviewed are **not** available through ACI. Available addresses of publishers are listed in the June "Current Reviews" each year. In most cases ACI can furnish addresses of publications added later.

For those members who cut apart this section for pasting on cards for card indexes, a limited number of complimentary reprints of the "Current Reviews" section are available from ACI headquarters on request.

this interesting paper. Reference is made to publications by R. Agent, V. N. Gornov, V. I. Murasov, I. M. Nenirovschi, E. Rausch, and Roghitki.

Design of concrete structures

LEONARD CHURCH URQUHART, CHARLES EDWARD O'ROURKE, and GEORGE WINTER, McGraw Hill Book Co., New York, 6th Edition, 1958, 546 pp., $8.00

An up-to-date revision of the popular text and reference book on reinforced concrete design. Improved over the 1954 edition and made current with the new 1956 ACI Building Code, including ultimate strength design with discussion, examples, and design aids. The column design section has also been revised for the new method of designing eccentrically loaded columns. The book has always been excellent for use as a text and practicing engineers will find the revision particularly helpful.

Elastic design method for structures without stress-load proportionality (in Hebrew)

A. ZASLAVSKY, *In the Field of Building, Bulletin* No. 48, Technion—Israel Institute of Technology (Haifa), 1958, 12 pp.

AUTHOR'S SUMMARY

Reviews structures which, although in the elastic range, do not exhibit a proportionality between load and stress. These are, in general: structures where the change in geometry due to deformation cannot be neglected; structures partially or fully prestressed; partially precast structures; those erected in stages; those with thermal or assembly stresses; structures strengthened to accommodate additional loads; composite structures; and structures with restraints coming into action only after initial deformation has taken place.

Materials

Normal and abnormal set of cement (in Dutch)

K. L. A. VAN DER LEEUW, *Cement* (Amsterdam), V. 9, No. 5-6, June 1957, pp. 205-206

Reviewed by JOHN W. T. VAN ERP

False set and flash set are two of the best known phenomena of abnormal set. The first is due to the fast hydration of gypsum which has been dehydrated by too much heat in the sintering process. False set can be overcome by continued mixing or stirring so that the flimsy gypsum network is broken up again and the normal set takes place undisturbed. Flash set is more complicated and can hardly be corrected if it has taken place. It is mostly due to the presence of too much calcium aluminate with too little gypsum as retarding agent. Both these phenomena are caused by inadequate control of the manufacturing process.

Research on high-turbulence mixed grout (Recherches sur le malaxage a "haute turbulence" des suspensions de ciment)

MICHEL PAPADAKIS, *Publications Techniques* No. 82-83 Centre D'Etudes et de Recherches de L'Industrie des Liants Hydrauliques (Paris), pp. 1-25; reprinted from *Revue des Matériaux de Construction*, No. 498, Mar. 1957

AUTHOR'S SUMMARY

A study has been made of the rheological characteristics of grouts produced with a "high-turbulence" mixer. The results obtained, in particular, with a viscosimeter of the coaxial type show that these grouts have a flow, and stability, which makes them eminently suited to injection work.

Microscopic study of portland cement clinker (La microscopie du clinker)

RENE ALEGRE and PIERRE TERRIER, *Publication Technique* No. 88, Centre D'Etudes et de Recherches de L'Industrie des Liants Hydrauliques (Paris), 44 pp.; reprinted from *Revue des Matériaux de Construction*, No. 501, June 1957

AUTHOR'S SUMMARY

The first part deals with the preparation of clinker for reflection microscope examination. The work then goes on to a macrographic and microscopic study of commercial clinker, with illustrations of the various aspects met with. The most frequent form of each constituent is given, together with the changes to which this is subjected due to the action of various factors. Numerical results enable determining the formation of some clinkers.

Pavements

Performance of doweled joints under repetitive loading

L. W. TELLER and H. D. CASHELL, *Public Roads*, V. 30, No. 1, pp. 1-24, Apr. 1958

Reports on comprehensive tests of full size models. Data were developed on the relationship between dowel diameter and load transfer capacity, and on the relation between slab

depth and dowel diameter required to transfer given percentages of applied load. Embedded length of dowel required for maximum load transfer was also studied. The effect of joint opening width on performance of dowels was tested under static load and repetitive loads up to two million cycles.

Field study of joint and crack resealing methods and materials

L. A. Fickes and C. C. Rhodes, *Bulletin* No. 166, Highway Research Board, 1957, pp. 1-15

Joints and cracks in a 19-year old, 5-mile stretch of concrete pavement on US 16 between Nunica and Fruitport, Michigan, were resealed under contract on a force account basis. Six different brands of hot-pour joint sealer were used.

After 2½ years of service, the various maintenance repairs have held up very well with the exception of material failures in two of the six brands of joint sealer used. The results of this work indicate that the widespread failure of joint seals in Michigan has been due partly to deficiencies in the sealing material and partly to inadequate cleaning and sealing operations.

Prestressed Concrete

Heat-insulating roof elements of prestressed concrete used for industrial buildings (in Rumanian)

M. Popovici, *Industria Constructiilor si a Materialelor de Constructii* (Bucharest), V. 8, No. 10, Oct. 1957, pp. 546-557

Reviewed by J. J. Polivka

Various structural members are thoroughly described and analyzed on the basis of research performed by author. Nine types were investigated by experiments and theoretical analysis: (1) thin slabs in prestressed concrete with insulating top slab of expanded slag concrete; (2) thicker slabs of expanded reinforced slag concrete with a row of circular holes (an alternative was tested with a double row of circular holes having smaller diameter); (3) hollow blocks of various cross sections assembled by typical post-tensioning methods with grouted joints; (4) reinforced concrete elements, either steam cured or pretensioned, with air spaces for thermal insulation; (5) elements of pretensioned lightweight concrete, consisting of 1¼-in. slab and edge ribs, with semirigid top insulation, ¾ in. thick; (6) cellular steam cured concrete cast between prestressed concrete ribs, with thickness equaling the full depth of the ribs; (7) an alternate of Type 6, thinner slab of cellular concrete cast partly on top of the ribs; (8) an alternate of Type 7, the slab of cellular steam cured concrete combined with pretensioned concrete ribs; (9) alternate of Type 8, both the lightweight concrete slabs and prestressed ribs being poured monolithically.

The problem of creep in slabs and disks subjected to prestressing in two directions (in German)

W. Swida, *Publications*, International Association for Bridge and Structural Engineering (Zurich), V. 16, 1956, pp. 469-484

Applied Mechanics Reviews Mar. 1958 (Phillips)

Deals with creep effects in plates and slabs prestressed in two directions. A system of two simultaneous first-order differential equations is derived and solved for the loss of tensile force in the cables. In the case of an eccentrically prestressed plate, the deflections are also determined.

Properties of Concrete

Gas concrete hardening without application of steam curing under pressure (Neautoklávovany plynobetón)

Jaromir Jambor, *Stavebnicky Časopis Slovenskej Akadémie Vied* (Bratislava), V. 6, No. 5, Nov. 1957, pp. 306-323; No. 6, Dec. 1957, pp. 361-374

Reviewed by J. Stork

Results of tests of properties of gas concrete made with aluminum powder and Czechoslovak cements and aggregates. The investigation was concentrated on determination of the effects of mix proportioning, aluminum powder dosage, concrete slump, fineness of grinding, pozzolans, and accelerating admixtures. Gas concrete was found equivalent and in some respects superior to orthodox wall and insulation material.

Alkali-aggregate reactions (in Dutch)

R. A. J. Bosschaert, *Cement* (Amsterdam), V. 9, No. 11-12, Dec. 1957, pp. 494-500

Reviewed by John W. T. Van Erp

The author enumerates many examples of concrete structures which have deteriorated due to alkali-aggregate reactions. The basic chemical reactions are analyzed. Various types of cracking patterns, spalling, efflorescence, swelling, and porosity are attributed

to alkali-aggregate reactions. The often recommended solution of using alkali-free cement is not accepted because it still leaves the concrete vulnerable to alkali attack from outside. Author recommends thorough chemical investigation and definition of the various silica, quartz, and other aggregates as an absolute necessity to solve the problem.

Rheological properties of concrete (O rheologických vlastnostech betonu)

Ivo Babuska, *Stavebnícky Časopis Slovenskej Akadémie Vied* (Bratislava), V. 7, No. 1, Feb. 1958, pp. 28-42

Reviewed by J. Stork

Critical review of theories of setting, hardening, and creep of concrete in effort to establish a mathematical expression of the function of creep and relaxation of concrete.

Studies of electrolysis between steel and concrete (Observations et remarques sur le potentiel de contact fer-ciment)

A. Nicol, *Publication Technique* No. 76, Centre D'Etudes et de Recherches de L'Industrie de Liants Hydrauliques (Paris), 49 pp.; reprinted from *Revue des Matériaux de Construction*, No. 487 and 489, Apr. and June 1956

Author's Summary

Should the cement, or ambient, medium allow the transmitting of electrical charges, a current will be set up in the mass, with the usual consequences: electrolysis and polarization, involving the risk of corrosion.

Every metal-cement junction must be considered as being capable of giving rise to an EMF, variable with composition and the conditions of preservation.

Structural Research

Studies (including tests) of thin shells with double curvature. Part I (in Rumanian)

I. Facaoaru, N. Galna, and D. D. Nicolescu, *Industria Constructiilor si a Materialelor de Constructii* (Bucharest), V. 8, No. 11, Nov. 1957, pp. 627-635

Reviewed by J. J. Polivka

Experimental studies of new type of thin shell roofs in reinforced concrete, standardized types, spanning 36 to 78 ft. Three-hinged precast arched sections have rise one-fifth of the span and a width of 3 to 6½ ft, with constant thickness 1⅝ in. Standard reinforcement is 1.12 percent of concrete area in longitudinal and 0.25 percent in transverse direction. Tests were made with quarter size models and also on natural size structure with average span of 59 ft. Deflections under various loading (uniform, partial, concentrated) and safety factors are recorded.

Impact tests of columns in reinforced concrete (in Serbian)

V. Andrejev and Z. Kostrencic, *Naše Gradevinarstvo* (Belgrade), V. 11, No. 12, Dec. 1957, pp. 282-288

Reviewed by J. J. Polivka

Tests were performed on two models of reduced dimensions: cross section 4¼ sq in; height 27 in.; resting on steel plate 4¾ in. square. Columns carry horizontal prismatic body of reinforced concrete 28 in. long, 7¾ in. wide, and 13 in. deep, monolithically interconnected and topped with a steel plate. Top body simulates the rigidity of supported floor structure. Hammer weighing 23 lb was dropped from five different heights, from 12 to 60 in. Two electric strain gages were installed and oscillations photographed in combination with Föppl's apparatus. Four hundred tests results are compared with theoretical analysis as developed by S. Timoshenko.

General

Should thin walls of concrete be reinforced? (Faut-il armer les parois minces en béton)

A. Vaissac, *Revue des Matériaux de Construction* (Paris), No. 507, Dec. 1957, pp. 362-363

Reviewed by Phillip L. Melville

The dangers of reinforcing thin (less than 80 mm) walls of concrete are discussed in view of corrosion danger, and subsequent concrete cracking, for the steel. The author concludes that without special precautions to waterproof the finished product, precast concrete in thin sections must have a concrete cover of at least 35 mm over the steel or the steel should be omitted.

Some economic aspects of the use of high strength reinforcing steel (in Dutch)

J. Middendorp, *Cement* (Amsterdam), V. 9, No. 3-4, Apr. 1957, pp. 132-134

Reviewed by John W. T. Van Erp

An analysis is presented of the various cost items: concrete in place, formwork, reinforcing steel, steel placement; on this basis a cost formula is derived. Total savings of 7-8 per-

cent are found by the use of high strength steel (f_s = 30,000 psi instead of f_s = 20,000 psi). Also considered are possible savings in the foundation cost by the use of smaller, lighter members, as well as some reduction in building height.

An automatic device to measure the time of set of cements (Un appareil automatique de mesure du temps de prise des ciments)

M. Papadakis and M. Minerbe, *Revue des Matériaux de Construction* (Paris), No. 505, Oct. 1957, pp. 287-289
Reviewed by Phillip L. Melville

The apparatus is a modification of the Vicat needle to obtain a continuous record of the set of cement. Electrically controlled pneumatic system is claimed to be reliable and well suited for both research and control testing.

Structures of Eduardo Torroja

Eduardo Torroja, F. W. Dodge Corp., New York, 1958, 208 pp., $8.50

An English translation of the autobiography of engineering accomplishments of Torroja. Thirty of Torroja's most impressive structures are discussed including bridges, churches, viaducts, stadiums, water towers, factories, dams, and hangars in prestressed concrete, wood, brick, and steel as well as reinforced concrete. The book is organized in five major sections: large shells, viaducts and aqueducts, special structures, steel and composite structures, and churches and chapels. Engineering details are given. Some alternate designs which were later discarded are shown. Includes 275 illustrations of completed projects, models, details, and projects under construction.

Calculation of temperature in mass concrete taking account of the ratio of hydration heat development as function of ambient temperature (Výpočet teploty v masívnych betónoch vzhladom na závislost rýchlosti vývinu hydratačného tepla od teploty prostredia)

Stavebnícky Časopis Slovenskej Akadémie Vied (Bratislava), V. 7, No. 1, Feb. 1958, p. 53; No. 2, Apr. 1958, p. 63
Reviewed by J. Stork

Analysis of influences on hydration heat development in order to establish an analytic method of computation of temperature in mass concrete structures with respect to the effect of the ambient temperature on the ratio of heat development. The problem is solved by means of method of finite differences.

PERIODICALS REGULARLY SCANNED FOR REVIEWS

The many requests for information concerning the source material of reviews appearing in the "Current Reviews" section indicate the desirabillty of publishing a list of those periodicals, bulletins, and proceedings being *scanned regularly* for articles dealing with the field of concrete. If all miscellaneous magazines, bulletins, reports, and books were included (which are reviewed only as they come to the attention of the voluntary reviewers or editors) the following list would cover well over 200 periodicals and/or publishing, educational, and research organizations. The following list includes publishers' addresses for convenience should correspondence with them be desired, since copies of articles or books reviewed in the Journal are *not* available through ACI. In most cases they can be obtained directly from the original publishers.

American Ceramic Society Bulletin—American Ceramic Society, 4055 North High St., Columbus 14, Ohio

AREA Bulletin—American Railway Engineering Association, 59 East Van Buren St., Chicago, Ill.

ASTM Bulletin—American Society for Testing Materials, 1916 Race St., Philadelphia 3, Pa.

Annales de L'Institut Technique du Batiment et des Travaux Publics—L'Institut Technique du Batiment et des Travaux Publics, 6 Rue Paul Valéry, Paris 16e, France

Annales des Ponts et Chaussees—Commission des Annales à l'École Nationale des Ponts et Chaussées, 28 Rue des Saints-Pères, Paris 7e, France

Applied Mechanics Reviews—American Society of Mechanical Engineers, 29 West 39th St., New York 18, N. Y.

Architectural Forum—Time & Life Bldg., 9 Rockefeller Plaza, New York 20, N. Y.

Architectural Record—119 West 40th St., New York 18, N. Y.

Australian Journal of Applied Science—Commonwealth Scientific and Industrial Research Organization, 314 Albert St., E. Melbourne, C 2, Victoria, Australia

Bauingenieur, Der—Reichpietschufer 20, Berlin, W 35, Germany

Bauplanung Bautechnik—Unter den Linden 12, Berlin NW 7, Germany

Bauzeitung, Die—Hospitalstrasse 12. Stuttgart N, Germany

Betongen Idag—Norsk Cementforening, Munkedamsveien 3b, Oslo, Norway

Beton i Zhelezobeton—State Library of U.S.S.R., Kalinin St. 3, Moscow G-19, Russia

Betonstein-Zeitung—Kleine Wilhelmstrasse 7, Wiesbaden, Germany

Beton-Teknik—Christians Brygge 28, Copenhagen V, Denmark

Beton- und Stahlbetonbau—Hohenzollerndamm 169, Berlin-Wilmersdorf, Germany

British Constructional Engineer, The—32 Southwark Bridge Road, London, SE 1, England

BRI Abstracts of Building Science Publications—Building Research Institute, 2101 Constitution Ave., Washington 25, D. C.

Building Research Reports, Danish National Institute of Building Research—Danish National Institute of Building Research, 20 Borgergade, Copenhagen K, Denmark

Building Research Station Digest—Building Research Station, Garston, Watford, Herts, England

Bulletin, Centre d'Etudes et de Recherches de l'Industrie des Liants Hydrauliques—Centre d'Etudes et de Recherches de l'Industrie des Liants Hydrauliques, 197 Boulevard Saint-Germain, Paris 7e, France

Bulletin de la Reunion Internationale des Laboratòries d'Essais et de Recherches sur les Materiaux et les Constructions—Reunion des Laboratories d'Essais et de Recherches sur les Materiaux et les Constructions, 12 Rue Brancion, Paris 15e, France

Bulletin of the Faculty of Engineering, Cairo University—Cairo University, Giza, Egypt

Bulletin of the Highway Research Board—Highway Research Board, 2101 Constitution Ave., Washington 25, D. C.

Bulletin of the Institution of Civil Engineers of Ireland—Institution of Civil Engineers of Ireland, 35 Dawson St., Dublin, Ireland

Bulletin del Instituto Tecnico de la Construccion y del Cemento—Instituto Técnico de la Construcción y del Cemento del Patronato Juan de la Cierva de Investigación Técnica, Apartado de Correos No. 2, Costillares, Chamartin, Spain, and Apartado de Correos No. 19,002, Madrid, Spain

Bulletin of the Iowa Engineering Experiment Station—Iowa Engineering Experiment Station, Iowa State College, Ames, Iowa

Bulletin of the University of Illinois Engineering Experiment Station—University of Illinois Engineering Experiment Station, 106 Engineering Hall, Urbana, Ill.

Cahiers du Centre Scientifique et Technique du Batiment—Centre Scientifique et Technique du Batiment, 4 Avenue du Recteur-Poincaré, Paris 16e, France

Cement—Herengracht 507, Amsterdam C, Netherlands

Cement and Lime Manufacture—14 Dartmouth St., London, SW 1, England

Cement och Betong—Kolga-huset, Postgiro 1808, Malmo, Sweden

Cemento, Il—Via Benedetto Marcello 2, Milan 405, Italy

Cemento Portland—Calle San Martin 1137, Buenos Aires, Argentina

Ceramic Abstracts—(see *Journal of the American Ceramic Society*)

Civil Engineer Corps Bulletin—Bureau of Yards and Docks, Department of the Navy, Washington 25, D. C.

Civil Engineering—American Society of Civil Engineers, 33 West 39th St., New York 18, N. Y.

Civil Engineering and Public Works Review—8 Buckingham St., London, WC 2, England

Commonwealth Engineer, The—349 Collins St., Melbourne, C 1, Australia

Concrete—400 W. Madison St., Chicago 6, Ill.

Concrete and Constructional Engineering—14 Dartmouth St., London, SW 1, England

Concrete Construction Magazine—139 N. Clark St., Chicago 2, Ill.

Concrete Products—79 W. Monroe St., Chicago 3, Ill.

Construction—92 Rue Bonaparte, Paris 6e, France

Constructional Review—14 Spring St., Sydney, N.S.W., Australia

Consulting Engineer—227 Wayne St., St. Joseph, Mich.

Contractors and Engineers—470 Fourth Ave., New York 16, N. Y.

Crushed Stone Journal, The—National Crushed Stone Association, 1415 Elliot Place, NW, Washington 7, D. C.

Deutsche Bauzeitschrift—Eickhofstrasse 14/16, Gütersloh, Germany

Engineer, The—28 Essex St., Strand, London, WC 2, England

Engineering—35 & 36 Bedford St., Strand, London, WC 2, England

Engineering Journal, The—2050 Mansfield St., Montreal 2, Quebec, Canada

Engineering News-Record—330 West 42nd St., New York 18, N. Y.

Genie Civil, Le—5 Rue Jules Lefebvre, Paris 9e, France

Geotechnique—Institution of Civil Engineers, Great George St., London, SW 1, England

Giornale del Genio Civile—Via Nomentana 2, Rome, Italy

Gradbeni Vestnik—Erjavceva 11a, Ljubljana, Yugoslavia

Handlingar (Proceedings), *Swedish Cement and Concrete Research Institute*—Svenska Forskningsinstitutet for Cement och Betong vid Kungl. Tekniska Hogskolan I, Stockholm, Sweden

Highway Research Abstracts—Highway Research Board, 2101 Constitution Ave., Washington 25, D. C.

Housing Research Papers, HHFA—Housing and Home Finance Agency, Division of Housing Research, Washington 25, D. C.

Hungarian Technical Abstracts—Hungarian State Technical Library, P. O. Box 12, Budapest 8, Hungary

Indian Builder, The—Ionic, off Arthur Bunder Rd., Colaba, Bombay 5, India

Industria Constructiilor si a Materialelor de Constructii—Revistele Tehnice ASIT, Str. Ioan Ghica nr. 3, Bucharest, Rumania

Industrial and Engineering Chemistry—American Chemical Society, 1155 Sixteenth St., NW, Washington 6, D. C.

Informes de la Construccion—Institute Técnico de la Construcción y del Cemento del Patronato Juan de la Cierva de Investigación Tecnica, Apartado de Correos No. 2, Costillares, Chamartín, Spain, and Apartado de Correos No. 19,002, Madrid, Spain

Ingenieria—National School of Engineering, University City, D. F., Mexico

Ingenieria Civil—Plaza de la Republica No. 55-602, Apartado Postal 10423, Mexico 1, D. F., Mexico

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Ingeniøren—Dansk Ingeniørforening, Ingeniorhuset, Vester Farimagsgade 29, Copenhagen V, Denmark

Journal of the Boston Society of Civil Engineers—Boston Society of Civil Engineers, 715 Tremont Temple, Boston, Mass.

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Journal of the Japan Society of Civil Engineers—Japan Society of Civil Engineers, No. 4, 2-Chome, Ohte-Machi, Chiyoda-Ku, Tokyo, Japan

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Polish Technical Abstracts—Centralny Instytut Dokumentacji Naukowo-Technicznej, Aleja Niepodleglosci 188, Warsaw, Poland

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Proceedings of the American Society of Civil Engineers—American Society of Civil Engineers, 33 West 39th St., New York 18, N. Y.

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Transactions of the Institution of Civil Engineers of Ireland—Institution of Civil Engineers of Ireland, 35 Dawson St., Dublin, Ireland

Transactions of the Japan Society of Civil Engineers—Japan Society of Civil Engineers, No. 4, 2-Chome, Ohte-Machi, Chiyoda-Ku, Tokyo, Japan

Travaux—6 Avenue Pierre 1er-de-Serbie, Paris 16e, France

VDI Zeitschrift—Verein Deutscher Ingenieure-Verlag GmbH, Ingenieurhaus, Prinz-Georg-Strasse 77, Düsseldorf, Germany

World Construction—24 W. Maple St., Chicago 10, Ill.

Zement-Kalk-Gips—Kleine Wilhelmstrasse 7, Wiesbaden 16, Germany

Zement und Beton—Oesterreichischen Betonvereins und Verein der Oesterreichischen Zementfabrikanten, Strohgasse No. 21 A, Vienna 3, Austria

Discussion of a paper by Charles S. Whitney:

Ultimate Shear Strength of Reinforced Concrete Flat Slabs, Footings, Beams, and Frame Members Without Shear Reinforcement*

By S. CONSTANTINESCU-CATUNESTI, JAIME DE LAS CASAS, W. T. MARSHALL, and AUTHOR

By S. CONSTANTINESCU-CATUNESTI†

The author suggests a method for applying ultimate strength design to footings. The total amount of steel in each direction is distributed unequally through the footing; the number of bars needed for shear is concentrated through the pyramid of rupture and the difference is placed outside of that pyramid. Fig. A(a) illustrates the solution. When the number of bars in zone a_2 is very different from the number of bars in a_1, it means that the flexural moments in the a_1 zones are not provided with consistent reinforcement. This occurs because the logical appearance based on Taylor and Thompson's *Treatise of Concrete* (1909 edition), which stated that "the bending moments decrease very rapidly," is incorrect. In a paper[10] issued in 1947, I found the distribution of flexural moments for a footing, considering that both the support and the footing are deformed under a concentrated load. Fig. A(b) shows the bending moments for a footing with the span, a = 4.00 m ($\sim$ 160 in.); depth, d = 33 cm ($\sim$ 13 in.); subgrade reaction, K = 8 kg per cu cm (27.7 lb per sq in. per in.); modulus of elasticity, E_c = 210,000 kg per sq cm ($\sim$ 2,980,000 psi).

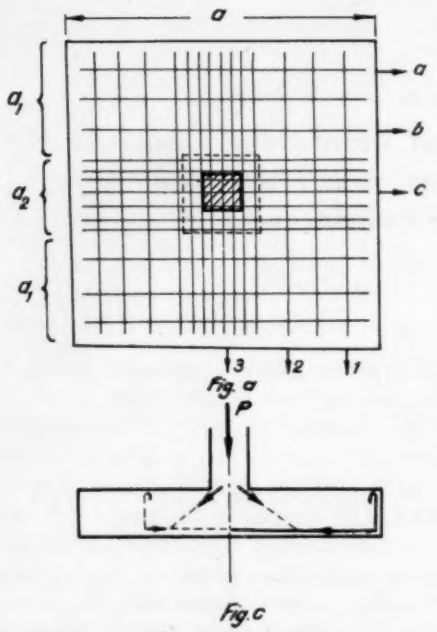
The flexural moment near the perimeter, on 1/10 of the span, is 0.65 of the center moment. It should also be noted that the work of Richart,[2] stated that the strains in the outer bars of square footings were similar in magnitude to those in the center bars, indicating that the full width of footing is effective in resisting bending moment. Corner areas in footings are also important as it is there that the torsional moments are generated by flexural moments in the diagonal direction.

It appears that Mr. Whitney's suggestion might be modified to say that the reinforcement needed for flexure may be spaced uniformly on the span a and enough supplementary steel placed through the pyramid of rupture to provide consistent shear strength.

It should be remembered that many codes[12-14] permit a reduction of the number of bars (15-20 percent) in the proximity of the perimeter only for flat slabs built with perimetrical beams.

*ACI JOURNAL, Oct. 1957, *Proc.* V. 54, p. 265. Disc. 54-15 is a part of copyrighted JOURNAL OF THE AMERICAN CONCRETE INSTITUTE, V. 29, No. 12, June 1958, *Proceedings* V. 54.

†Professor of Reinforced Concrete, Railway Engineering Institute of Romania, Bucharest, Romania.



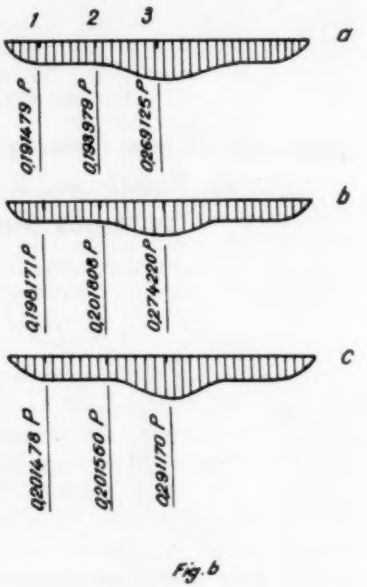


Fig. A(a)—Footing reinforcement. Fig. A(b)—Distribution of moments for footing. Fig. A(c)—Suggested reinforcement anchorage

To guarantee that the horizontal component of the shearing force will be resisted by the reinforcing bars passing through the pyramid of rupture, the bars can be anchored as shown in Fig. A(c).

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11. Henry, F. D. C., *The Design and Construction of Engineering Foundations*, London, 1956.
12. STAS 1546-50, "Calculul Elementelor de Beton Armat," Bucharest, 1952.
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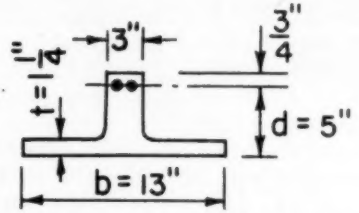
By JAIME DE LAS CASAS*

In the section discussing simple beams under concentrated loading (pp. 285-290), the writer would like to call attention to Eq. (2):

$$v_u = 50 + 0.26 \frac{M_u}{d^2} \sqrt{\frac{d}{l_a}}$$

*Member American Concrete Institute, Civil Engineer, Lima, Peru.

TABLE A—COMPARISON OF DATA AND COMPUTATIONS



Cross section of T-beams tested by Al-Alusi

	$\frac{a}{d}$	$\frac{d}{l_s}$	v_u*, psi Eq. (2)	v_{cr}, psi Tests	v_{cr}/v_u
p = 1.48 percent Reinforcement— 2 # 3 bars	2.0	0.50	177	228	1.29
	3.36	0.30	149	203	1.37
	4.0	0.25	140	189	1.35
	4.18	0.24	138	185	1.34
	4.45	0.22	136	174	1.28†
p = 2.7 percent Reinforcement— 2 # 4 bars	3.45	0.29	210	250	1.19
	4.0	0.25	197	205	1.04
	4.5	0.22	184	178	0.97
	6.5	0.15[5]	166	198	1.19
	7.44	0.13[4]	159	189	1.19†
p = 4.2 percent (over-reinforced) Reinforcement— 2 # 5 bars	5.50	0.18	217	186	0.86
	6.54	0.15[3]	186	176	0.95
	7.76	0.13	193	183	0.95

*In computing v_u it was considered that f_c' = 4000 psi and f_y = 53,000 psi.
†Flexural failure.

This formula was checked with the test results reported by A. F. Al-Alusi in "Diagonal Tension Strength of Reinforced Concrete T-Beams with Varying Shear Span," ACI JOURNAL, May 1957, *Proc.* V. 53, pp. 1067-1077.

The writer found good agreement between Al-Alusi's T-beam tests and Eq. (2) provided that M_u/d^2 was computed as for rectangular beams with tensile reinforcement only. If so, it could be inferred that with the same tensile reinforcement the ultimate shear strength is similar in T-beams or rectangular beams. However, further study is necessary to verify this criterion.

Some of the principal relations between Eq. (2), computed as above, and Al-Alusi's tests are shown in Table A.

As can be noted in Table A, when the percentage of reinforcement increases v_{cr}/v_u decreases. This is due to the fact that the T-beams tested were 3 in. wide and, therefore, the larger bars had a very close spacing, especially in T-beams with p = 4.2 percent where the clear spacing was less than 1 in. and maybe less than one diameter. The relation between bar spacing and ultimate shear strength was mentioned by Mr. Whitney in several sections of the paper.

The writer believes that this paper is another important contribution by Mr. Whitney to the ultimate strength design method.

By W. T. MARSHALL*

The writer was interested in Mr. Whitney's paper based on the tests carried out by F. E. Richart because the writer performed a series of tests on square isolated column bases in 1938-39, the publication† of the results being delayed until 1944 because of war conditions.

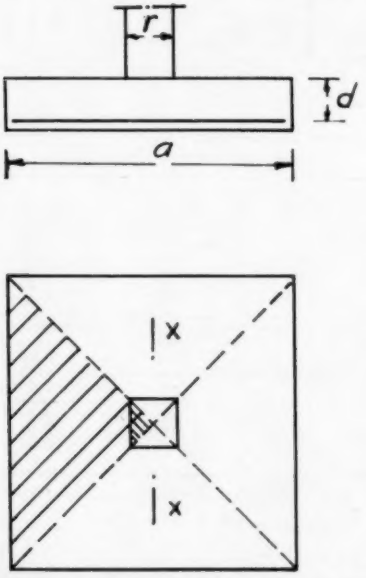


Fig. B—Calculating bending moment across a column base. M_{xx} moment due to force from shaded triangles

These tests were made on 72 bases each 24 in. square and 4 in. deep, the load being applied through a steel block 4 in. square. The reinforcement was placed in the bottom with ½-in. cover, equal areas running in each direction, the percentage varying from 0.12-1.46. The slabs were supported on a sand base which was uniformly compacted, thus differing from the spring supports used in Richart's tests. Previous experiments on small scale mortar bases had shown the writer that the distribution of pressure under a base varies with the type of bed. In the case of a compacted sand, a pressure distribution which is assumed to be parabolic with a maximum value at the center and zero at the four corners agreed with the test results. The reinforcement could be divided into two main classes: (1) that with adequate anchorage, and (2) that with an anchorage less than that specified by the British Standard Code of Practice.

The method of calculating the bending moment across a column base in Britain differs from that given on p. 274 of the paper. British practice is to assume that one-quarter of the downward load from the column is taken by each triangle shown in Fig. B thus giving $M = P/4\ [\bar{x} - (r/3)]$ where $\bar{x}$ is the distance from the center of the base of the center of pressure of the upward force on one triangle. In the case of uniform pressure distribution $\bar{x} = a/3$ but with a parabolic distribution $\bar{x} = 0.3a$.

The results of the tests showed that in the case of bars with adequate anchorage the failing load agreed reasonably well with that obtained by calculating M_u on Hajnal-Kónyi's theory, but making some allowance for the

*Regius Professor of Civil Engineering, University of Glasgow, Glasgow, Scotland.

†Marshall, W. T., "Experiments on Reinforced Concrete Column Bases," *Journal*, Institution of Civil Engineers (London), Mar. 1944.

tension taken by the concrete in the case of small percentages of reinforcement. The failing load was obtained by equating the M_u thus obtained to $P/4\,[0.3a - (r/3)]$.

If however an analysis is attempted on the lines given by Mr. Whitney, the results do not give $v = 100 + (3M_u/4d^2)\sqrt{d/l_s}$. Taking the values for M_u given on p. 275 for under-reinforced beams with adequate anchorage and taking $M_u = os_b l_b\,[1 - (pf_y/1.7f_c')]$ in the case of bond failures the analysis with $v = k_1 P/[4r\,(r + d)]$ is shown in Fig. C and D.

Fig. C, which applies in the case of bars with adequate anchorage, gives $v = 100 + 2.0\,(M_u/d^2)\sqrt{d/l_s}$ and Fig. D, which applies for bond failures, gives $v = 75 + 1.3\,(M_u/d^2)\sqrt{d/l_s}$. There are some interesting features about these results: (1) The value of v, namely 100, when $M_u = 0$, is the same as in the Richart tests in the case of Fig. C. (2) The slope in both Fig. C and D is much greater than the ¾ value given by Mr. Whitney.

One criticism against the results shown in Fig. C is that most of the results have a value of $(M_u/d^2)\sqrt{d/l_s}$ less than 100 whereas those given in the paper have an average value of approximately 250. The percentage of reinforcement used in most isolated column bases is, however, small—averaging about 0.25 percent. This low value would correspond to $(M_u/d^2)\sqrt{d/l_s}$ approximately equal to 80, the approximate average value in Fig. C.

The difference between the writer's results and that presented in the paper may be due amongst other things to the use of the sand bed instead of the springs but this also represents more closely the practical conditions.

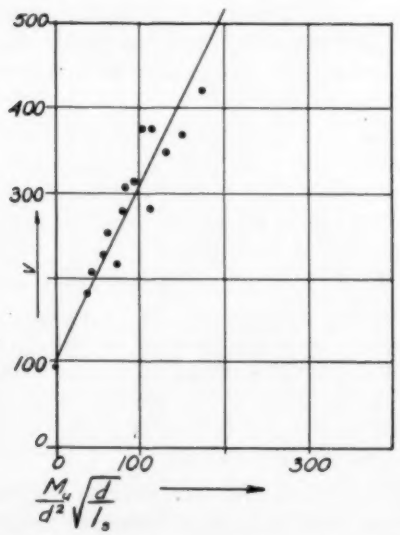


Fig. C—Analysis for under-reinforced specimens with adequate anchorage

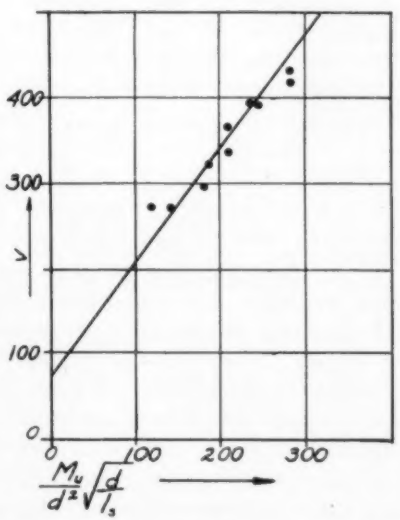


Fig. D—Analysis for specimens with bond failures

The writer feels however that there is too big a difference between the coefficients of $(M_u/d^2)\sqrt{d/l_s}$ in the two series of tests for him to have great faith in the relationship $v = 100 + \frac{3}{4}(M_u/d^2)\sqrt{d/l_s}$.

AUTHOR'S CLOSURE

Since the paper was prepared about a year ago, a considerable body of additional test data has become available, covering a wider range of variables. Continued study of this data indicates that the equations can be improved by including other parameters and that several suggestions require modification. Unfortunately it is not yet possible to make definite recommendations for design in spite of intensive analyses made to date. This work is being continued by the joint ACI-ASCE committee on shear and diagonal tension (Committee 326), with the hope that a report can be prepared this year for discussion in 1959 so that a method for shear design consistent with the ultimate strength method may be included in the next edition of the ACI Code.

Committee 326 was organized in 1950 to develop methods to resist shear and diagonal tension consistent with the new ultimate strength design methods which were finally approved in 1956. Twelve meetings of the committee have been held in addition to numerous subcommittee meetings. As the first phase of this assignment, it undertook to determine the shear strength of members without shear reinforcement, and test programs were investigated to provide the necessary information regarding the behavior of slabs, beams, and frame members. These tests have since been covered by individual published reports. A number of additional test programs have also been sponsored by various agencies such as the American Iron and Steel Institute, Bureau of Public Roads, Corps of Engineers, Concrete Reinforcing Steel Institute, Portland Cement Association, and Association of American Railroads, some of which have not yet been published.

In spite of the great accumulation of test data available in 1950 from shear tests of beams, the data now being used by the committee as a basis for a new method of design has been developed since that time. This is because it is now recognized that design for shear strength should be based on parameters that had not been considered important in the previous tests.

In October, 1955, as a result of some recent shear failures in reinforced concrete structures, the committee made recommendations to ACI Committee 318, resulting in new code provisions which were incorporated in the 1956 ACI Code. These consisted primarily in a reduction of the maximum shear permitted without shear reinforcement, an increase in the amount of shear reinforcement for rectangular frames and continuous beams, and additional requirements for reinforcement near points of contraflexure.

Subsequent tests and studies indicate that these rules will produce reasonably safe structures when used with the ultimate method of design, although further extension of flexural steel and shear reinforcement might be desirable. It is the author's feeling that the same rules should be applied to T-beams,

even though they may have appeared to behave better in structures than rectangular sections. Most concrete beams are continuous or restrained and they need stirrups in all cases to tie in compressive reinforcement whether or not the compressive strength of the steel is needed.

It has of course become obvious that the conventional shear formula, $v = V/bjd = kf_c'$, is not suitable for use because the shear strength is not a simple function of concrete strength, but depends also on the amount of flexural reinforcement, the moment-shear relation which can be represented by a function of M/V_d or the so-called depth to shear-span ratio, and the span to depth ratio.

Because the value of j in the conventional shear equation is usually assumed to be a constant, 0.875, it can best be omitted and the unit shear calculated as the average value for the full depth, d.

Several meetings of a subcommittee held this past winter have resulted in progress and an agreement on a number of points. To coordinate analysis of all of the pertinent test programs and arrive at a comprehensive solution, the test data is being processed on computing machines. The weakness of previous solutions, including those in the October paper lay in the fact that the data was based on parameters of too limited range and sometimes on the wrong parameters. Only in the last few years has attention been paid in the test programs to diagonal cracking loads and the effect of shear span. Only in the last few months has the span-depth ratio been separated as a parameter. With the advent of the ultimate strength method of design the effect of high concrete and steel strengths and higher percentages of steel must also be evaluated.

In the case of beams, the committee is agreed that the first critical diagonal tension cracking load should be considered as the ultimate. For the longer beams (with a shear span greater than about four times the depth) failure is sudden following the first diagonal crack. As the span-depth ratio decreases, the mode of failure changes to shear-compression and the ultimate strength becomes increasingly higher compared to the first diagonal cracking load. This excess strength should probably not be utilized in members without shear reinforcement, but when shear reinforcement is provided, it may be practical to provide a relatively smaller amount in the very short beams. This becomes important in some structures, as for instance, bridge pier frames carrying heavy concentrated loads.

The committee also agrees that shear strength of beams is a function of concrete strength, percentage of steel, and moment-shear ratio but not the yield strength of the steel. In a beam, when the yield strength of the steel is reached, a flexural failure occurs and the shear strength is then not significant. The influence of concrete strength appears to be a function of $\sqrt{f_c'}$.

The formulas for shear strength should be applicable to beams under concentrated loads, uniform loads, continuous beams, and to beams subjected to direct compression or tension as in frame members.

The critical section of a beam is not necessarily the section at the face of column or the section of maximum shear. Studies of new test data are still being made to determine the best form of equation and location of critical section to correlate the various tests.

The effect of the M/V ratio, or shear span, is complicated by the influence of stress concentration due to the load. This is illustrated by the behavior of test beams loaded at the center through a column stub. In the longer beams the first diagonal crack appeared about half-way between the column stub and the support, or at a distance about $2d$ from the load. The shear span at the critical section was therefore at least $2d$ less than the maximum shear span at the load point.

The shorter beams failed at a higher load with the crack appearing at an angle of roughly 45 deg from the face of the column stub. In that case the shear span at the critical section may have been the same as for the longer beam, but the strength was greater, due to the influence of the column load.

It appears necessary to differentiate between the shearing strength of a beam near the support or load points and that in the portions more remote, perhaps at a distance $2d$ or more from the load concentrations. This will also affect the location and amount of reinforcing needed.

The case of flat slabs is somewhat different from that of beams. The shear strength is a function of p, f_c' and the shear span, but also may be affected by the yield strength of the steel or flexural moment capacity. Relatively thick slabs may fail in shear before f_y is reached, but in thin slabs f_y may be reached in rods near the column before full yield line flexural strength develops.

In the case of tests of 6 ft square slabs 6 in. thick with flexural bars uniformly spaced, the failure occurred in shear below the yield line flexural strength except in the more lightly reinforced slabs. Since the shear strength increases with the percentage of steel, the obvious way to balance shear and flexural strengths is to move some of the steel from the edge of the slab to central portion, thereby increasing the shear strength without decreasing the flexural strength. The difficulty with that idea is that, so far as a very narrow band is concerned, it does not appear to work. Tests made at the Portland Cement Association laboratory last winter on 6-ft slabs with steel concentrated through the pyramid of rupture, a band about 18 in. wide, indicated that both shear strength and flexural strength were practically the same as with the same total area of steel uniformly spaced for the 6 ft width. The stiffness of the slab is increased and the cracking decreased by concentrating the steel near the column. Since the shear strength is increased by increasing the percentage of steel, it still remains to be determined what width of band is effective and whether or not this band is narrow enough to permit balancing shear and flexural strength without increasing total steel. Latest tests indicate the band within the pyramid of rupture is too narrow to control shear strength.

It has been found that shear reinforcement for flat slabs is inefficient and expensive and usually should be avoided if possible.

The writer wishes to thank the discussors for their helpful suggestions.

Discussion of a paper by J. J. Shideler:

Lightweight-Aggregate Concrete for Structural Use*

By E. W. BAUMAN and AUTHOR

By E. W. BAUMAN†

The investigation reported by Mr. Shideler makes another important contribution for the designing engineer who uses lightweight structural concrete. Although many of the properties of lightweight concrete included in this investigation were previously investigated,[14-17] the data developed probably represent more reliable information on present-day conditions, as the aggregates included are more typical of currently produced lightweight materials. Actually, some aggregates in the earlier investigations included several experimental materials and a few that are no longer produced. Also, since this is the first important contribution on lightweight concrete coming directly from the Portland Cement Association, the report carries special significance in that it registers a definite recognition by the association of the importance which this newer type of concrete now assumes. Mr. Shideler handled the investigation, and the report as well, in a very commendable manner.

Since only one expanded slag—Aggregate No. 6—was included in the investigation, and the other lightweight aggregates were all of a clay or shale type, the title of the report is a trifle misleading. Probably this is not too important, except that when the reference is included in future bibliographies on lightweight concrete, researchers may be misled to expect a broader coverage of lightweight aggregates, per se, than is actually included. Inasmuch as expanded slag is currently being produced at 21 plants, with an annual output of approximately 3,000,000 tons, it is regretted that more samples of this type lightweight aggregate were not tested.

Properties of the slag lightweight concrete, except for compressive strength, compare favorably with concretes of expanded clay and shale aggregates. From the viewpoint of this observer, strengths—certainly beyond the range of 3000 or 4000 psi—are not too important in lightweight concrete. There is no problem in proportioning expanded slag concrete mixes that will easily yield results well within this strength range. Actually, the strength of expanded slag concrete at a given cement factor, can be materially increased by the use of small amounts of natural sand in the fine aggregate—and with little increase in weight. Unfortunately this rather important feature was not investigated by Mr. Shideler. It is hoped that in the future, investigators

*ACI JOURNAL, Oct. 1957, *Proc.* V. 54, p. 299. Disc. 54-16 is a part of copyrighted JOURNAL OF THE AMERICAN CONCRETE INSTITUTE, V. 29, No. 12, June 1958, *Proceedings* V. 54.

†Member American Concrete Institute, Managing Director, National Slag Association, Washington, D. C.

other than those in the laboratory of the National Slag Association—where this point, along with other pertinent findings, has been clearly established —may include something on this phase of the problem.

Since lightweight concrete will, in most cases, be used in the interior of buildings, the other properties such as light weight, shrinkage, acoustic properties, modulus of elasticity, creep, heat transmission, and fire resistance are certainly as important as compressive strength, if not more so. All of the foregoing, with the exception of fire resistance, have been included in one or another of the investigations reported, and what can be expected from lightweight concrete in regard thereto has been rather well established.

FIRE RESISTANCE

The fire resistance of lightweight concrete is important because of its favored use for interior construction of walls, floors, beams, joists, etc., especially so when it is realized that the concrete structure may serve as a hospital, apartment, school, or other public building. Yet we know much more about fire resistance of heavy concrete than of lightweight concrete. The fact that Mr. Shideler made no reference to fire resistance in his report, is certainly no indication that PCA itself does not recognize this as being of prime importance.

Actually, there have been fire resistance tests conducted on lightweight concrete at the Underwriters' Laboratories, sponsored by PCA. My main purpose in this discussion is to focus attention on the results obtained and thereby effect a tie-in with Mr. Shideler's paper. Thus architects and designing engineers using the paper will not be disappointed to find that reference to this very important property of lightweight concrete has been omitted. Although the fire tests referred to were confined to a 4½-in. *reinforced concrete* slab and beam floor, the results obtained may be considered applicable to plain lightweight concrete as well.

The report itself was published by the Underwriters' Laboratories as Retardant No. 3390-7, dated Aug. 26, 1952. Only expanded slag aggregate was used. The coarse slag weighed 43 lb per cu ft, dry, loose, and had a maximum size of ½ in.; the fine slag weighed 60 lb per cu ft. Note that this compares well with the weight of the slag included in Shideler's investigation, which he reports as 44.2 lb for the coarse, and 64.2 for the fine. The concrete had 8.1 sacks of cement per cu yd; weight of dry concrete was 104 lb per cu ft; and compressive strength at 28 days was 3840 psi. These properties also compare favorably with reported data. Results of fire endurance for this 4½-in. floor slab yielded a 4-hr rating, which is the only 4-hr rating for 4½-in. floor slab of any concrete materials listed (according to the January, 1957 listing of fire resistance ratings of the National Board of Fire Underwriters).

This would indicate that lightweight concretes, particularly those made of expanded slag lightweight aggregate, will provide excellent fire resistance ratings. Whether all lightweight aggregates can impart this high fire en-

durance property to the concrete as well, is a matter of importance and since so little information is available on this item, it appears in order to recommend that any future investigations dealing with properties of lightweight concrete be so planned as to devote a major portion thereof to fire resistance tests. Until that time, the fire tests referred to may be used as a guide to what can be expected from lightweight concrete.

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16. "Lightweight Aggregate Concretes," Housing and Home Finance Agency, Aug. 1949.

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AUTHOR'S CLOSURE

The author appreciates the comments and additional information provided by Mr. Bauman. Our work on structural quality lightweight aggregate concrete is still continuing, and this first published report gives only a portion of the total data obtained on this subject. Coefficients for use with ultimate strength design involving lightweight concrete were included in a paper by Eivind Hognestad,* and it is expected that shear strength data for reinforced concrete beams will be published soon in the ACI JOURNAL.

As indicated by Mr. Bauman, the Portland Cement Association considers fire testing of concrete of prime importance. A few tests on lightweight concrete have been conducted at the Underwriters' Laboratories and a new Fire Research Center constructed by the Portland Cement Association will provide greatly expanded facilities for testing and research on various types of concrete and concrete structural members.

Addition of natural sand to improve strength and workability is sometimes used with other lightweight aggregates as well as with expanded slag. Mr. Bauman recognizes the amount of work represented by this paper, and that the inclusion of another variable, such as natural sand, although possibly desirable, immediately doubles the testing program.

Our data on expanded slag aggregate indicate that a high quality concrete can be produced from this material when proper recognition is given to aggregate grading, mixing procedure, and the use of entrained air.

*"Confirmation of Inelastic Stress Distribution in Concrete," *Proceedings*, ASCE, V. 83, ST 2, Paper 1189, Mar. 1957. Also issued as *Development Department Bulletin* D15, Portland Cement Association.

Discussion of a paper by Jack E. Rosenlund:

Construction of the Dallas Memorial Auditorium*

By ERIC C. MOLKE and AUTHOR

By ERIC C. MOLKE†

The separation of a dome-shaped roof into 16 pie-shaped segments which structurally act in pairs, so that the resulting eight arches are pushing outward against 32 cantilever bents, is most interesting because it allows construction with eight uses of a rotating formwork, while a conventional dome would have to be formed as one monolith.

It is difficult, however, to see that a true dome construction of 300-ft span should not have been more economical, if also considering the difference in costs between the light columns necessary to support a true dome and the extremely heavy cantilever bents necessary to resist the arch thrust of the segmental cover.

Roughly it appears that the quantities used for the vertical legs of the cantilever bents are about 12 lb reinforcing steel and 8 in. of concrete per sq ft of covered dome area on a 300-ft diameter. It can be deduced that of these quantities about 8 lb reinforcing and 5 in. of concrete could be saved by using a true dome, since the cantilever framing is principally designed for bending stresses due to the cantilever and the outward thrust of the arch ribs. This indicates that a penalty of at least $2 per sq ft should be applied to the structure as described when comparing its roof costs with those of a full concrete dome.

It should be further remembered that for the comparatively small and uniform dead load of a true dome rather light and inexpensive shoring and forming can be built. A dome shoring of 4 x 4-in. posts at 8-ft centers, nailed together with light bracing, is suitable and is described for a dome of similar height conditions in *Construction Methods and Equipment* (January, 1954). Such falsework has been used successfully on various dome constructions in this country and abroad and will require approximately 10 fbm of lumber per sq ft of plan area, including form ribs and sheeting. Camber requirements are small when using a prestressed dome base ring, and decentering can easily be accomplished by removing simple wedges under the posts. Considerable equipment loads can also be hung to such a dome.

*ACI Journal, Oct. 1957, *Proc.* V. 54, p. 329. Disc. 54-17 is a part of copyrighted Journal of the American Concrete Institute, V. 29, No. 12, June 1958, *Proceedings* V. 54.

†Member American Concrete Institute, Chief Engineer, Prestressing Research and Development, Inc., San Antonio, Tex.

In the following engineer's estimate for price comparison, a price of $2.50 per sq ft plan area has been used for the dome formwork, which is higher than indicated by actual construction data available to the writer.

	Monolithic dome		Segmental cover	
Concrete, including base ring	10 in. per sq ft	$0.85	9.7 in. per sq ft	$0.77
Reinforcing steel	5 psf @ 0.18	0.90	8 psf @ 0.18	1.44
Prestressing wire in base ring	0.7 psf	0.35		—
Formwork	Wood (single use)	2.50	Steel (eight uses)	1.25
Extra for vertical leg of supporting bents, with segmental cover		—		2.00
Total cost of roof structure above columns, per sq ft of plan area		$4.60		$5.46

Quantities and costs for the segmental cover as used in the above comparison have been taken from a discussion on this subject in *Engineering News-Record*, July 19, 1956. It is not clear whether these quantities cover all items equivalent to the monolithic dome and base ring of 300-ft diameter, as they seem small by comparison, but the results of the tabulation should justify doubts of the author's statement, p. 330, "that total cost was about half of that estimated for either a full concrete dome or a structural steel dome."

The application of a rotating formwork with the segmental construction procedure as it is so ably described in this paper should, however, be of great interest to the concrete dome designer. There is no doubt that such procedure, in conjunction with a temporary system of segmental arch ribs, should make concrete construction more competitive for large circular roofs.

AUTHOR'S CLOSURE

I wish to thank Mr. Molke for his interesting thoughts on the Dallas Memorial Auditorium and its design. It is gratifying to know that the subject initiated his comments.

I have reviewed Mr. Molke's letter with the consulting engineers, Ammann and Whitney, and our conclusions are very nearly the same. The advantages of a monolithic concrete dome, as outlined by Mr. Molke, are well taken and these advantages were considered in the design of the Dallas Memorial Auditorium. In any structure, the cost and function are the prime considerations to a client. In order to obtain true cost comparisons, two designs necessarily must be made, one for a monolithic dome and one for the segmental roof.

The design selected was based on a thorough study of function and cost. The architects and engineers are still convinced that the City of Dallas' interests are better served by the design selected. If a roof alone were the prime consideration, then a monolithic dome might have more merit. However, the author believes the function of the building deserves other considerations.

Had the design been a full monolithic dome (of 300-ft diameter), the shape of the building would have changed materially. If this change in shape had been acceptable, the moving of the curved roof support to the outside wall line would have materially increased the height of the building and also the volume. Had the roof been designed as full monolithic dome so as not to increase the height, facilities of the building (projection booths, control rooms, mechanical, heating and air-conditioning space) would have been greatly limited, or would have resulted in a much flatter roof, which would have, in the author's estimation, increased at least the material costs.

Mr. Molke compares "light columns" supporting a monolithic dome with extremely heavy cantilever bents required to resist arch thrust and support the segmental roof. The light columns to which Mr. Molke refers would also carry half the seating slab load, floor loads, wall loads, mezzanine floor (supporting mechanical equipment and press rooms), ramps, full lateral load due to wind and the 24-ft cantilevered canopy. It is the writer's opinion that there would be little difference in Mr. Molke's light columns and the columns of the present design. For this reason, the $2.00 per sq ft penalty that Mr. Molke attaches to the segmental roof appears to be an unjust figure.

A point of interest in Mr. Molke's comments is the cost of formwork for a monolithic dome. The $2.50 per sq ft for single-use formwork, forming a double curved roof 90 to 110 ft in the air all at one time, seems extremely low to the writer. This is especially true where the shoring would be required to bridge large openings in the first floor.

Boyd Anderson, of Ammann and Whitney, has stated concerning the two designs that there is no magic and little difficulty in either design, but to get true facts on costs of either design would involve contractors' bids of the two designs. Regardless of Mr. Molke's interesting comments, the writer is still convinced the design selected resulted in the lowest cost for a building which served the desires of the city and is the most functional of all designs considered.

Discussion of a paper by G. C. Ernst:

Ultimate Torsional Properties of Rectangular Reinforced Concrete Beams*

By FRITZ LEONHARDT, W. T. MARSHALL, and AUTHOR

By FRITZ LEONHARDT†

The results reported by Professor Ernst are most valuable and will help in developing formulas for the ultimate torsional strength.

I would like to know if it was possible to observe where the first crack in the concrete began.

These tests varied the transverse steel by varying the number and spacing of the #2 ties. The number of the longitudinal bars (four corner bars) remained the same, only the size varying from #3 to #5.

The investigation could be supplemented in a valuable way by additional tests with more longitudinal bars of smaller size and at small spacing along the side faces of the beam. I am fairly certain that the ultimate torsional resistance would be greatly influenced by such bars because the principal tensile stresses are larger at the sides than at the corners and diagonal tension needs a narrow net of orthotropic reinforcement if diagonal bars are to be omitted.

By W. T. MARSHALL‡

The writer read with interest Professor Ernst's paper but feels that he must disagree strongly with the statement made on p. 351: "With no transverse reinforcement it is evident that the maximum torque is limited to the first term of Eq. (1) which is the maximum torque for a plain (unreinforced) section with v_{max} limited to the diagonal tension value."

When any material is twisted, shear, tensile, and compressive stresses of equal magnitudes are set up on planes perpendicular to or at 45 deg to the axis of twist. The type of failure which occurs is governed by the ability of the material under test to resist stresses of each of these types. Thus with a mild steel rod, failure takes place along the plane perpendicular to the axis of twist because in a mild steel rod the ultimate shear strength is less than the ultimate tensile or compressive stresses. With a thin-walled steel tube on the other hand, failure occurs by buckling along a 45-deg helix because of the low compressive stress of the thin steel plate. With plain concrete,

*ACI JOURNAL, Oct. 1957, *Proc.* V. 54, p. 341. Disc. 54-18 is a part of copyrighted JOURNAL OF THE AMERICAN CONCRETE INSTITUTE, V. 29, No. 12, June 1958, *Proceedings* V. 54.

†Member American Concrete Institute, Consulting Engineer, Stuttgart, Germany.
‡Regius Professor of Civil Engineering, University of Glasgow, Glasgow, Scotland.

failure also takes place along a 45-deg helix (as shown in Type 1, Fig. 7) because concrete is weaker in tension than in shear or compression. Consequently the strength of an unreinforced concrete torsion member depends on the tensile strength of the concrete. If, however, v_{max} is calculated from Eq. (1) using the value of α given it will be found that v_{max} is much greater than the tensile strength of plain concrete. The error is caused by taking the value for α corresponding to an elastic state of stress.

If the concrete behaved as an elastic material then the failure of an unreinforced specimen would start by the development of a crack at the midpoint of the longer side. The failure however takes place suddenly, the cracking occurring over the whole section simultaneously.

This points to a plastic stress distribution at failure. If this is so then it can be easily shown that

$$T = vb^2d\,(0.5 - b/6d)$$

The writer analyzed* a number of investigations into the torsional resistance of plain concrete specimens on the plastic equation given above and found that the value for v was approximately equal to the tensile strength of plain concrete.

Analysis on the elastic lines given by Professor Ernst is not in the writer's opinion completely valid for plain concrete.

AUTHOR'S CLOSURE

Dr. Leonhardt's comments on the increasing need for information on the ultimate torsional strength of reinforced concrete is heartily concurred with. In addition to the further tests proposed by Dr. Leonhardt, future investigations should include combined bending and torsion, and also the transfer of torsional moments into columns as flexural moments.

With regard to the starting point of the first crack, the beams with the greatest amount of transverse steel exhibited cracking that began at middepth of the 12-in. faces and spread toward the top and bottom of the beams. For beams with less transverse steel, it was difficult to be certain as to the starting point because of the rapidity of crack formation.

Professor Marshall clearly describes the effect of differences in shearing, tensile, and compressive strengths in resisting torsion. Assuming the tensile strength to be controlling, it is evident that a material must have sufficient plastic tensile deformation to distribute the ultimate tensile strength uniformly from the outer surface to the center to develop the full plastic torque represented by $T = vb^2d\,[0.5 - (b/6d)]$, with v equal to the ultimate tensile strength. Although concrete has a certain amount of plastic tensile deformation, it is extremely small and should result in failure at a torque closer to the elastic state than the plastic state. The average v resulting from the plastic

*Marshall, W. T., "The Torsional Resistance of Plastics with Special Reference to Concrete," *Concrete and Constructional Engineering* (London), Apr. 1944.

equation for the beams without ties is about 188 psi. If a semiplastic equation for rectangular beams is approximated by proportioning between the elastic and plastic states in the same ratio as exists for the circular section with a parabolic stress-strain relationship, the value of v would be about 224 psi. Although direct tension tests were not made for the mix used, the tensile strengths of equivalent mixes reported in other investigations in the United States have averaged close to 350 psi. The assumption of a linear elastic state at ultimate torque for the beams with no ties provides an average value for v of 332 psi, which is in better agreement with the tension tests made in the United States than either the fully or partially plastic states of stress. However, Professor Marshall's investigations of plain concrete in torsion show good agreement with the plastic equation, which would seem to point to the possibility of a difference between the cements used in the United States and Scotland. The higher tensile deformability implied by agreement with the plastic equation would be a very desirable quality.

Dr. Leonhardt's and Professor Marshall's discussions have been most helpful in emphasizing several important aspects of torsion when applied to reinforced concrete beams.

Discussion of a paper by K. E. McKee and E. I. Fiesenheiser:

A Critical Look at Slab Design Methods*

By KUANG-HAN CHU, PAUL ROGERS, and AUTHORS

By KUANG-HAN CHU†

A valuable contribution has been made by Messrs. McKee and Fiesenheiser in providing a comparison of ACI slab design methods with results based on yield-line theory. However, to use their data properly, the writer believes that the following points should be clarified.

1. As shown by the curves, the moments based on yield-line theory are closer to those based on ACI Method 1 than those based on Method 2. This is primarily because of the fact that both the moments based on yield-line theory and those based on ACI Method 1 are *average* moments of the *whole slab* while the moments based on ACI Method 2 are *average* moments in the *middle strip* only. To make a fairer comparison, the following paragraph in the ACI Code with regard to Method 2 should be noted:

"The average moment per foot width in the column strip should be $\frac{2}{3}$ of the corresponding moments in the middle strip."

Thus the average moment of the whole slab (since column strip and middle strip each equals $\frac{1}{2}$ of the width of the slab) is $\frac{1}{2} \times \frac{2}{3} + \frac{1}{2} \times 1 = 0.833$ of the middle strip moment. For example, for a square slab, $a/b = 1$, four edges pinned, the moment coefficient given by ACI Method 2 is 0.050. Therefore, the average moment for the *whole* slab based on ACI Method 2 is $0.833 \times 0.050 = 0.0416$. This is exactly the same as the coefficient given by ACI Method 1, $0.33 \times \frac{1}{8} = 0.0416$, which is also the same as the one given by the yield-line theory. Other coefficients can be compared the same way.

2. It can be shown that close agreements are found for all curves except for slab moments in the *short span* when the *long sides* are simply supported while the *short sides* are either simply supported or fixed. In such cases, ACI Method 2 gives much lower moments then either ACI Method 1 or the yield-line theory. However, it should be noted that ACI Method 2 was never intended for ideally simply supported edges, as negative moments are given for all discontinuous edges.

The 1940 Joint Committee Report stated: "These recommendations are intended to apply to slabs isolated or continuous, supported on four sides by walls or beams, in either case built monolithically with the slabs."

*ACI JOURNAL, Nov. 1957, *Proc.* V. 54, p. 397. Disc. 54-21 is a part of copyrighted JOURNAL OF THE AMERICAN CONCRETE INSTITUTE, V. 29, No. 12, June 1958, *Proceedings* V. 54.

†Member American Concrete Institute, Associate Professor, Department of Civil Engineering, Illinois Institute of Technology, Chicago, Ill.

No such positive statement has been found in the current ACI Building Code.

For those who are interested in the theoretical background of the ACI methods, it might be worth pointing out that ACI Method 2 was based on the papers by Westergaard and Slater in 1921[30] and Westergaard in 1926[28] and ACI Method 1 was based on the papers by DiStasio and Van Buren in 1936[24] and Bertin, DiStasio and Van Buren in 1945.[15] The writer would like to congratulate the authors for bringing to the attention of the profession the necessity of reviewing the background of the specifications from time to time.

By PAUL ROGERS*

The authors have presented an interesting and useful method of curve plotting which enables the reader to easily compare the different analytical and design methods.

A study of these charts reveals that there is a wide gap between ACI Methods 1 and 2, which are included in the ACI Building Code. When these methods, particularly Method 2, are compared with either the elastic or yield-line theory, one finds a discrepancy too large to be ignored.

The writer has endeavored for many years to bring about a modernization of the present code provisions for two-way slabs, or, until this can be achieved, to eliminate Method 2 as being unsafe, uneconomical, and unrealistic.[16]

The writer's Cases No. 1 to No. 9 (Fig. A) indicate, in detail, the major variations of moment coefficients in the short direction only when computed by the elastic theory and by ACI Method 2.

The charts of Fig. A include both positive and negative moments. The supports are assumed as unyielding, and Poisson's ratio is equal to zero.

The authors rendered a useful service by presenting their graphical method, and calling attention to the inadequacy of Method 2 of the ACI Code.

*Member American Concrete Institute, Consulting Engineer, Chicago, Ill.

Author's closure appears on p. 1182.

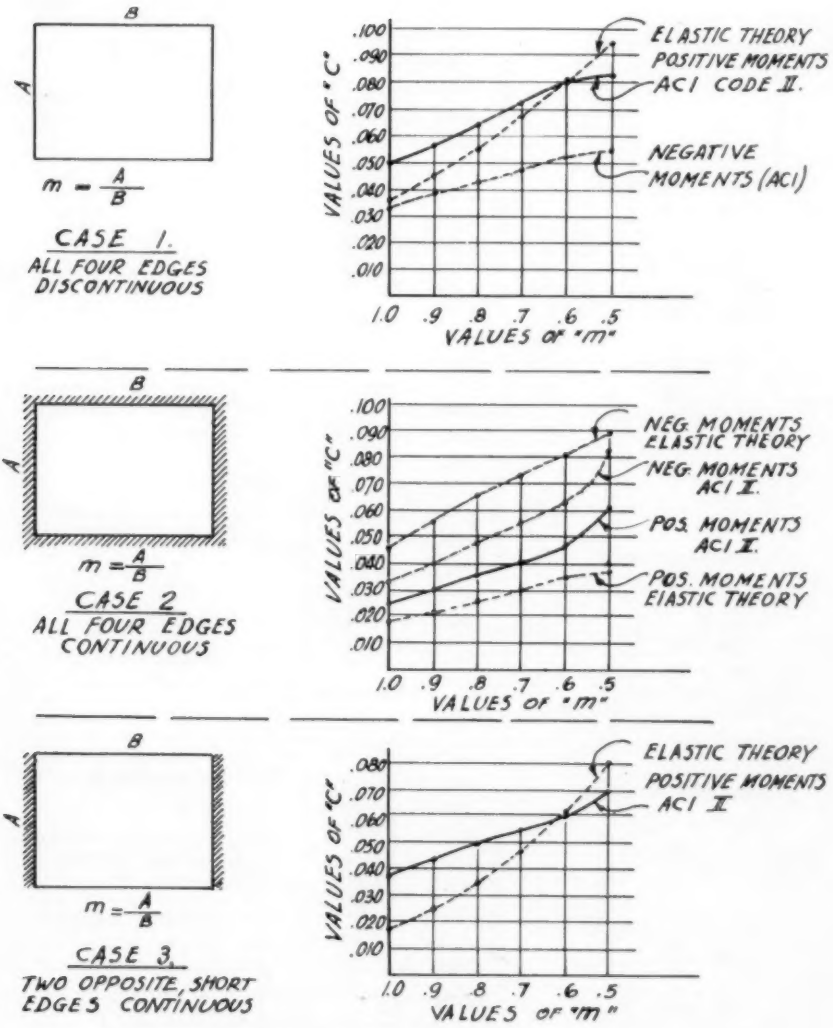


Fig. A—Variations of moment coefficients

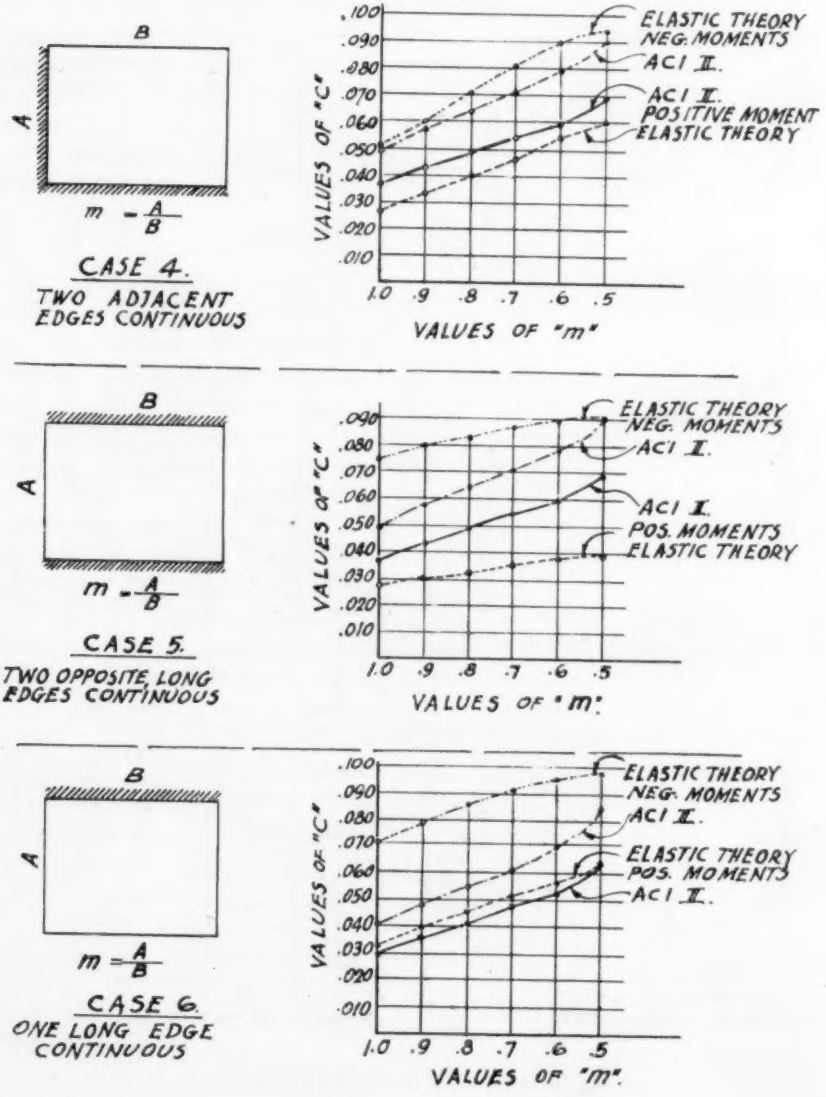


Fig. A—Variations of moment coefficients

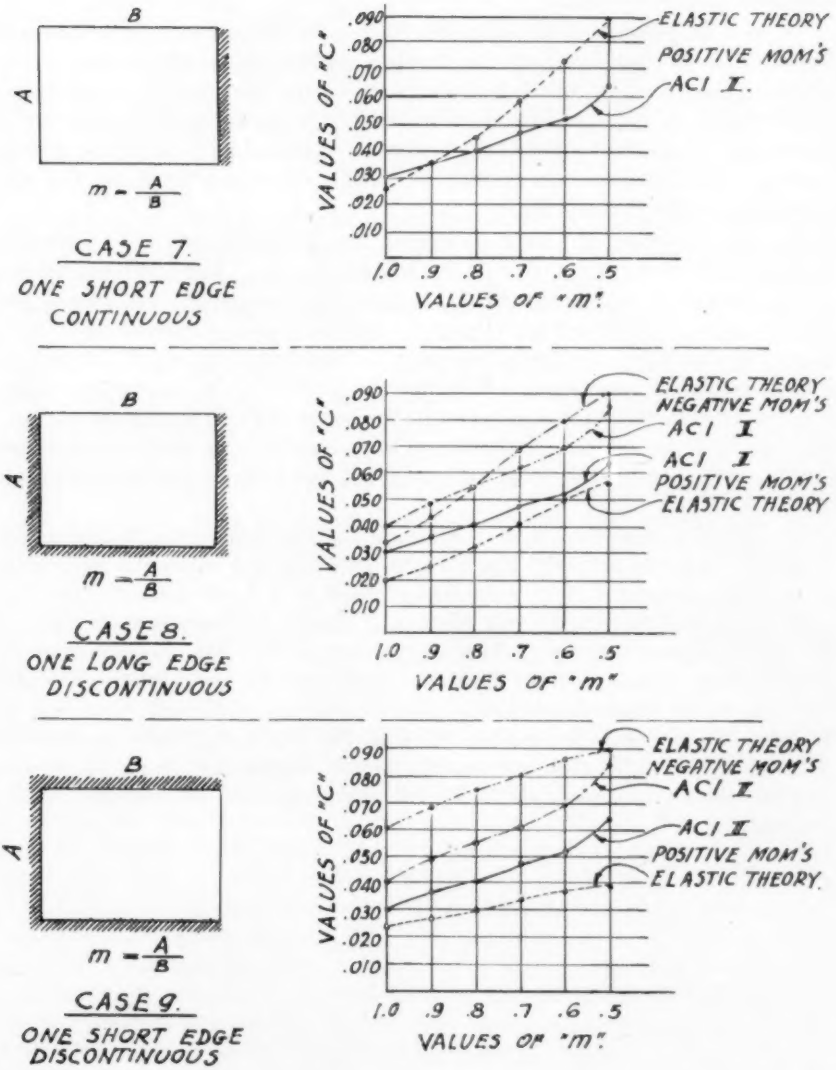


Fig. A—Variations of moment coefficients

AUTHORS' CLOSURE

The authors wish to thank Dr. Chu and Mr. Rogers for their interest in and contributions to the paper.

As Dr. Chu points out, the moments used in connection with ACI Method 2 were based on the requirements for the middle strip. This assumption, which is conservative, was introduced for two reasons: first, in some design offices the steel requirements over the middle strip are used for the entire width and, secondly, it allowed the analyses to be conducted in a more general fashion. The authors believe that Dr. Chu's comments have clarified this limitation of their presentation.

Dr. Chu's second comment concerned the limitation of ACI Method 2 to slabs built monolithically with the walls or beams. The 1940 Joint Committee limitation was known to the authors prior to publication. Since this limitation is not included directly or by reference in the current ACI Code, many, if not most, designers interpret a "discontinuous edge" to include simply supported conditions. Whether the limitation be on support conditions or on side ratios, as mentioned in the paper, it is obvious that the published limitations for ACI Method 2 are not complete. As shown in the paper unsafe designs can result from application of the code, as currently written, for this method.

Mr. Roger's presentation of his results for the nine cases is interesting. It should be noted that Mr. Roger's interpretation of a "discontinuous" edge is a simple support and a "continuous" edge as a fixed support. On that basis his comparison with ACI Method 2 should be considered in light of Dr. Chu's second comment. It should be noted that Mr. Roger's cases although more extensive than the four considered by the authors, are also limiting rather than all-inclusive. Any number of additional cases can be introduced by considering partial fixity at the edges, e.g., through negative moments resisted by torsional rigidity of the supporting beams at discontinuous edges or by interrelations with adjoining slabs and beams at continuous edges.

Discussion of a paper by William Shelson:

Bearing Capacity of Concrete*

By TUNG AU, D. CAMPBELL-ALLEN, W. G. PLEWES, MAURICE ROYER, and AUTHOR

By TUNG AU†

In the analysis of bearing test results presented in the paper, the author pointed out that since the testing blocks were generally bedded on heavy steel platens during the tests, the results are not necessarily representative of shallow blocks bedded on soil or other material with relatively low modulus of elasticity. This fact tends to restrict the scope of application of the test results as well as the conclusion on effect of depth-width ratio of the testing blocks. It is questionable, therefore whether the present test results should be accepted as sufficient evidence to revise the allowable bearing stresses permitted by Section 305 of the ACI Building Code (ACI 318-56) without further tests of more flexible blocks embedded on elastic foundation.

The report of the Joint Committee on Standard Specifications for Concrete and Reinforced Concrete‡ in 1940 adopted the following formula for minimum area A of footing or pedestal:

$$A = \left[\frac{f_b}{0.25\, f_c'}\right]^2 A'$$

where A' is the area of the column, f_b the allowable bearing stress on concrete in the column, and f_c' the ultimate compressive strength of concrete in the supporting member. It was stated, however, that "in all cases the concrete supporting member should be of such quality that the ratio of f_b to $0.25 f_c'$ does not exceed $1\frac{1}{2}$." In other words, although the formula is in effect identical to the curve based on Bauschinger's test data with a factor of safety of 4, the application to the determination of bearing pressure has been limited to the cases for which the ratio A/A' is equal to or less than $(1.5)^2$ or 3.375. Such restriction can be attributed to the uncertainty regarding the influence of large footing area on the concrete bearing pressure under the column when the footing itself is embedded on soil. In view of the low limiting value for A/A', this type of empirical formula does not serve the purpose it was intended to, and the variation of bearing pressure can be approximated by a

*ACI JOURNAL, Nov. 1957, *Proc.* V. 54, p. 405. Disc. 54-22 is a part of copyrighted JOURNAL OF THE AMERICAN CONCRETE INSTITUTE, V. 29, No. 12, June 1958, *Proceedings* V. 54.

†Member American Concrete Institute, Visiting Associate Professor of Civil Engineering, Carnegie Institute of Technology, Pittsburgh, Pa.

‡Joint Committee on Standard Specifications for Concrete and Reinforced Concrete, "Recommended Practice and Standard Specifications for Concrete and Reinforced Concrete," June 1940, Section 870.

straight line relation in this range. Hence, the allowable bearing stresses in the ACI Building Code are no more conservative than the provision in the Joint Committee Report except that, in the former, the limit of A/A' has been rounded off to 3.

For the cases of footings or pedestals embedded on bedrock, however, the test results presented and correlated by the author indeed indicate that, within the range of tests, the allowable concrete bearing stresses may be increased with an increase in the A/A' ratio. Unfortunately, even for this type of problem, the test data do not include points at lower ratios of A/A' which occur more often in actual practice. In this range, inference has been drawn from Bauschinger's test data on sandstone which, the author contended, cannot be applied to concrete without serious reservations. In examining Fig. 10, it is debatable whether the bearing pressure on concrete at $A/A' = 1$ should be taken as high as that indicated by the dotted line.

TABLE A—COMPARISON OF FORMULAS

$R = \frac{A}{A'}$	$4 \times f/f_c'$			
	Eq. (1)	Eq. (2)	Eq. (3)	ACI Code
1	1.00	1.00	0.86	1.00
2	1.23	1.26	1.08	1.25
3	1.39	1.44	1.24	1.50
4	1.52	1.59	1.36	1.50
6	1.71	1.82	1.56	1.50
8	1.86	2.00	1.72	1.50
10	2.00	2.15	1.85	1.50
20	2.47	2.71	2.33	1.50
30	2.77	3.10	2.67	1.50
40	3.02	3.42	2.94	1.50
50	3.23	3.68	3.17	1.50
60	3.41	3.92	3.36	1.50
70	3.58	4.12	3.54	1.50
80	3.72	4.31	3.70	1.50
90	3.85	4.48	3.85	1.50
100	3.98	4.64	3.99	1.50

One may agree with the author's opinion on the approximate shape of the curve outside the range of test results, but it is difficult to follow his reasoning in choosing an exponential function as given by Eq. (1) for allowable bearing stress. For $R = A/A'$,

$$f = 0.25 f_c' R^{0.3} \quad \text{.....(1)}$$

If the main purpose of the proposed formula is to provide a permissible bearing stress in accord with ACI Building Code at the lower end of the curve and to give a closer agreement with the test curve for higher ratios of A/A', there seems to be no need to replace the formula recommended by the Joint Committee Report which can be rearranged as follows:

$$f = 0.25 f_c' \sqrt[3]{R} \quad \text{.....(2)}$$

On the other hand, if the proposed formula is intended to reduce the allowable bearing stress for lower ratios of A/A' as well as to increase the allowable stress at higher ratios of A/A', the proposed formula has not gone far enough. One way of achieving this result and avoiding the exponent of 0.3 in the formula is to adjust the constant coefficient in Eq. (2). The following formula is an example:

$$f = 0.215 f_c' \sqrt[3]{R} \quad \text{.....(3)}$$

A comparison of these formulas is made for various ratios of A/A', and the results of f/f_c' multiplied by an assumed factor of safety of 4 are tabulated in Table A. It is obvious that all points computed by Eq. (3) for A/A' less than 8 lie below the dotted line in Fig. 10, while some of the points by Eq. (1) for the same range are not necessarily so, particularly since the dotted line itself may tend to dip further down near $A/A' = 1$. At higher ratios of A/A', points computed by Eq. (1), (2), and (3) all lie below the test curve in Fig. 11, and Eq. (1) and (3) give practically identical values.

By D. CAMPBELL-ALLEN*

Three separate investigations during the last 13 years have been carried out at the University of Sydney on mortar and concrete cylinders loaded axially by circular steel punches over a part of their areas. The ratio of concrete area to loaded area has been in the range from 1.8:1 to 28:1 with the majority of observations having a ratio of less than 8:1. These tests may therefore help to fill one of the gaps pointed out by the author.

The tests on mortar specimens, summarized in Table B, were made in 1945 using a 3:1 sand-cement mix with a W/C ranging from 0.62 to 0.50 by weight. Tests were made at about 7 days age (to speed results) and so represent low strength material. Each result quoted is the mean of three tests on similar specimens. The control tests were carried out on 4 x 8-in. cylinders at the same age as the bearing tests.

TABLE B—LOADING TESTS ON MORTAR CYLINDERS

Set	Diameter of punch, in.	Diameter of cylinder, in.	Height of cylinder, in.	Bearing pressure at failure, psi	Mortar cylinder strength, psi
1	1.0	4.1	2.2	4690	1070
	1.0	4.1	4.4	6080	
	1.0	4.1	6.2	6310	
	1.0	4.1	8.0	5820	
2	1.5	2.0	4.0	3040	2230
	1.5	4.1	4.0	5150	
	1.5	6.0	4.0	6990	
	1.5	8.0	4.0	7510	
3	1.0	2.0	4.0	1850	800
	2.0	4.1	8.0	1970	
	3.0	6.0	12.0	1980	
	4.0	8.0	16.0	2030	
4	1.0	4.1	8.0	9750	1720
	1.5	4.1	8.0	5470	
	2.0	4.1	8.0	3580	
	3.0	4.1	8.0	2690	

The concrete test results (Table C) were obtained in 1955 and 1956 as part of an investigation into anchor-zone stresses in prestressed concrete and represent results on high strength concrete. The number of test results used to obtain the mean and the standard deviation of test results is included in the table, which indicates that the testing variation was small for Set 6. Failure in all cases occurred by the formation of a cone (with a vertical angle of about 40 deg) followed by splitting of the cylinder into three or four pieces separated by radial cracks. A few tests using steel cones of about the same shape as the concrete failure cone showed that the load required to split the cylinder after the cone formed was much less than the load required to form the cone, thus indicating that failure follows closely the formation of the cone.

The results have been plotted in Fig. A. It appears that tests in which the depth/diameter is greater than 1:1 lie approximately on a straight line for the

*Member American Concrete Institute, Senior Lecturer in Civil Engineering, University of Sydney, Sydney, Australia.

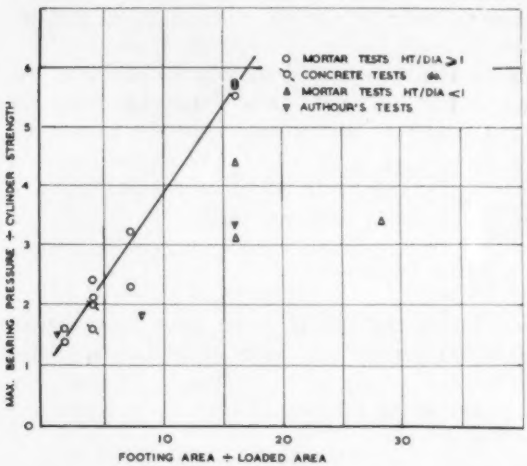


Fig. A—Concentric loading tests on mortar and concrete cylinders

range of footing area/loaded area from 1.8 to 16, and the values of bearing pressure are everywhere greater than the author's results for comparable footing areas. The change of shape from square to circular cross section may be in some measure responsible for this increase. In three cases, the depth/diameter ratio is less than 1:1, and in these cases a substantial reduction in bearing pressure was observed. This is contrary to the author's observation even though the test conditions were similar in that the specimens were bedded on the steel plate of the testing machine.

The author's tests are all on centrally loaded footings, while ACI 318-56 permits considerable eccentricity. Some light on the effect of eccentricity is given by a small number of tests in the 1945 series in which 4 in. diameter cylinders were loaded through 1 in. diameter discs with various eccentricities. Fig. B shows the results, which indicate that for the maximum eccentricity for which any stress increase is permitted by ACI 318-56 the bearing pressure is only 52 percent of the pressure when loading is concentric. Nevertheless,

TABLE C—LOADING TESTS ON CONCRETE CYLINDERS*

Set	No. of tests	Type of test	Bearing pressure, psi		Cylinder strength, psi	
			Mean	Standard deviation	Mean	Standard deviation
5	3	Punch loading	13,870	1880		
	3	Compression			8620	350
	3	"Brazilian" tension			514	104
6	8	Punch loading	11,180	340		
	6	Compression			5640	132
	5	"Brazilian" tension			463	10

*6 x 12-in. cylinders; 3 in. diameter punch.

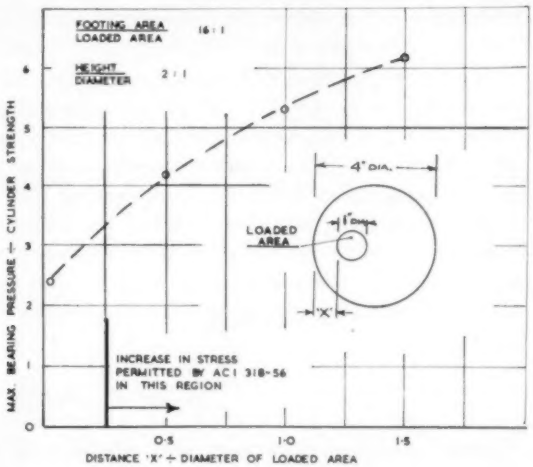


Fig. B—Eccentric loading tests on mortar cylinders

the bearing pressures are still substantially above those permitted by the ACI Code with a factor of 4. On the other hand if a similar reduction were obtained for the author's test conditions, the design curve proposed by the author would no longer be conservative.

The case of the cylinder loaded through a concentric punch has been analyzed by the theory of elasticity and experimental evidence has been obtained in support of the calculated stress distributions. It has also been shown that the stress distribution remains elastic (at least for high strength concrete) up to 80 percent or more of the failing load. This elastic analysis has however not been successful in predicting failing loads, since the tensile strain reached in the concrete has been shown to be of the order of four times that obtained in a simple tension test. A theory of failure accounting for this marked increase in tensile strength in material subjected to biaxial stresses has not yet been established. The case tested by the author has been analyzed elastically by Guyon,* and it would be interesting to know whether a similar increase in tensile strength occurs in the case of the square footings.

The tests were done by final year civil engineering students, W. S. Butcher and R. B. Cooper (1945); D. J. Douglas and N. S. Trahair (1955); J. Graham and N. M. Hawkins (1956) under the direction of H. J. Vogan and the writer.

By W. G. PLEWES†

It seems that the present allowable stresses for base plates on concrete footings founded on rock are based on a small amount of experimental data. The author has made an excellent contribution which points the way to higher

*Guyon, Y., "Contraintes dans les Pièces Prismatiques Soumises à des Forces Appliquées sur Leurs Bases, au Voisinage de ces Bases," *Publications*, International Association for Bridge and Structural Engineering (Zurich), V. XI, 1951, pp. 165-226.

†Building Structures Section, Division of Building Research, National Research Council, Ottawa, Canada.

allowable stresses where the footing area/loaded area ratio is high but more evidence is needed as he suggests.

The curve proposed as a substitute for the ACI curve is for footings having a depth/width ratio of 1:1. Meyerhof's data (Fig. 10) indicate that there are a whole family of curves for different depth/width ratios. It is difficult to say from the present data whether the effect of varying this ratio can be ignored for the usual ratios of bearing area/footing area and the author's assumptions in this regard are not clear. What for instance would be the effect of increasing the depth/width ratio to 1.5 where the footing area to loaded area ratio is small? From Fig. 12 it appears that the curve might fall below the ACI and proposed curves so that the desired factor of safety would be reduced.

By MAURICE ROYER*

In the author's timely and interesting article, he points out that the results of Bauschinger's tests may be approximately represented by the formula

$$f = f_c' \sqrt[3]{\frac{A}{A'}}$$

which may be written

$$f = f_c' R^{0.33}$$

where $R = A/A'$.

He then plots a curve (Fig. 11) representing the results of his tests and those of Meyerhof's tests.

He suggests that the formula

$$f_c = 0.25 f_c' R^{0.3}$$

is a reasonable and more realistic working formula with a factor of safety of 4. This formula is practically the same as that from Bauschinger's tests, with a factor of safety of 4.

The writer believes that, in view of the asymptotic characteristic of the curve of Fig. 11, a hyperbolic formula would be more consistent with the test results and would be of a simple application.

It will be found that the formula

$$f = f_c' \left(5 - \frac{20}{R - 5}\right)$$

is much closer to the test curve for all values of R above 10 than that sug-

*Member American Concrete Institute, Consulting Engineer, Quebec, Canada.

gested by the author. The writer therefore would suggest that if a load factor of N is desired, a working formula of the form

$$f_c = \frac{f_c'}{N}\left(5 - \frac{20}{R - 5}\right)$$

should be used for values of R above 10, and that a formula of the form

$$f_c = \frac{1.25}{N} f_c'$$

should be used for values of R up to 10, to conform more closely with the test results.

AUTHOR'S CLOSURE

The discussions by Messrs. Au, Campbell-Allen, Plewes, and Royer provide a valuable supplement to the data presented in the paper.

The selection of a suitable design formula depends to some extent on personal judgment and experience. It should be noted that Bauschinger's formula is based on the compressive strength of a cube-shaped test piece. Hence, this formula would not be valid if f_c' were taken to be the compressive strength for a 6 x 12-in. cylinder.

Elastic stresses in the square footings were calculated using Guyon's equations. It was found that for the loads required to cause failure of the footings, calculated elastic tensile stresses were greater than tensile strengths obtained from briquet tests. This result may not be significant since the equations do not provide an exact solution, nor do they include any allowance for plastic flow.

Discussion of a report by ACI Committee 711:

Proposed Revision of ACI Standard 711-53

Minimum Standard Requirements for Precast Concrete Floor and Roof Units*

By MICHAEL ALEXANDER, P. O. FREEMAN, and COMMITTEE

By MICHAEL ALEXANDER†

The presentation would be clarified by defining some terms. For example, several references are made to "factory methods" and "factory control." But surely the existence of a building called a factory doesn't assure a high standard of quality, since the concrete can vary just as much as can the competence, efficiency, or integrity of the managers. How about precasting on the job site? When can this qualify as factory control? It seems that what is needed is a clear statement of what procedures constitute "factory methods." It appears that a vague term has been used in the interest of brevity.

The term "topping" as used in Section 403 is also applied by some engineers to a field-placed floor slab over joists, such as described in Section 408. "Topping" is apparently intended to apply only to placement over a continuous concrete surface such as that presented by a series of contiguous channel slabs. It is not clear, however, why a shear stress of $0.02f_c'$ is allowed at the plane of construction in Section 403, while the corresponding value for precast joist floor design (Section 408), is permitted to be $0.03f_c'$.

This is further confused by the fact that in Section 403 the topping is required to be "thoroughly bonded to the surface of the precast unit." (It is hoped that the definition of satisfactory bonding will be in the body of the standard rather than in a footnote.) But Section 408, where the higher shear stress is permitted, requires no such special bonding procedure.

In Section 408, it is not clear whether the horizontal shear stress v_h is to be computed on the total contact surface between joist and slab, or only on the horizontal contact surface at the top of the joist. This is of interest when the slab concrete is weaker than the joist concrete.

It would also be helpful if every presentation of a proposed ACI standard were accompanied by a bibliography referring to test data or theoretical reasoning upon which the various provisions are based. Where no such data exist, it should be so stated, so that the practicing engineer can decide for himself how much reliance (whether on the conservative or liberal side) to place on

*ACI JOURNAL, Dec. 1957, *Proc.* V. 54, p. 185. Disc. 54-24 is a part of copyrighted JOURNAL OF THE AMERICAN CONCRETE INSTITUTE, V. 29, No. 12, June 1958, *Proceedings* V. 54.

†Member American Concrete Institute, Associate, Severud-Elstad-Krueger Associates, New York, N. Y.

criteria which have not been demonstrated in the laboratory. I would be very much interested, for example, in knowing whether there are test data upon which the recommendations in Section 801, referring to the lateral transfer properties of hollow-core units, are based.

By P. O. FREEMAN*

DESIGN, SECTION 407

This section establishes a method of determining the value of n for use in designing lightweight aggregate concrete and cellular concrete. We would draw to the committee's attention that when cellular concrete is manufactured by the "chemical" process, using aluminum powder to produce the rise of the material, it is impractical to cast 6 x 12-in. cylinders. The rise of the material is greatly inhibited by the relatively large area of mold surface in relation to the cross section of material attempting to rise. Specimens manufactured in cylinder molds would therefore have a greater density and a less uniform structure than material made in large molds in the production of reinforced slabs. For this reason, it is standard practice to do compression tests on 6-in. cubes.

Early in our research program, we manufactured many cylinders by turning the material on a lathe, and tested these cylinders in comparison with 6-in cubes to determine the ratio of strength values obtained from cylinder tests and cube tests. It was found that the 31 lb per cu ft material, having a compressive strength of about 500 psi, provided a cylinder:cube ratio of approximately 0.98. The 44-lb material, having a compressive strength of approximately 900 psi, provided a ratio of the order of 0.92. We have therefore used only cube tests in our control testing and have used the readings so obtained and a value of 40 percent f_c' for design calculations.

Due to the impracticability of test cylinders, we have employed a somewhat unconventional method of determining n. We have performed a large number of load-deflection tests and have calculated back from these tests to determine the value of n from observed deflections. We think this method is actually more closely applicable for design of reinforced slabs than is the method described in Section 407. This is proven by the fact that when a certain method for calculating the moment of inertia of the cross section is correlated with a value of the modular ratio found from deflection tests, in which the calculations employ that moment of inertia, the resulting designs can always be tested to provide an actual observed deflection closely related to the calculated deflection.

We therefore suggest that Section 407 be revised to include provision for the determination of n from the load-deflection curves of typical slabs from various manufactured series.

*Member American Concrete Institute, Research and Design Engineer, Siporex, Ltd., Montreal, Quebec.

DESIGN, SECTION 401

This section requires that the allowable design stresses shall conform to the appropriate sections of ACI 318-56. It has been our experience that with welded mats and a heavy anticorrosion treatment on the steel, which adheres more strongly to the steel than would concrete, the bond values may be increased considerably without affecting the deflection or ultimate strength characteristics of the slabs. This has been confirmed by performing pull-out tests, using both treated and untreated rods.

It is presumed that the increase in bond results from a combination of the increased diameter of reinforcing rod (due to the anticorrosion treatment), and from the autoclave curing.

It is therefore suggested that Section 401(a) be related to Section 101(c), where the peculiarities of a material or a process indicate that variation from the requirements of ACI 318-56 could be permitted.

DESIGN, SECTION 404

It has been our experience, confirmed by long-term tests, that autoclaved cellular concretes suffer less than one-tenth as much deflection due to shrinkage and plastic flow as do moist cured or low pressure steam cured concretes of conventional aggregate, lightweight aggregate, or cellular concrete.

In view of the many occasions which have arisen over the years where deflections of this type have developed in conventional precast or cast in place floors, the committee might give some consideration to controlling this type of deflection, either under Section 104 or under Sections 501 and 502.

INSTALLATION, SECTION 701

It is suggested that the committee might consider including, under this section, minimum end bearing for precast slabs.

COMMITTEE CLOSURE

As pointed out by Mr. Freeman, difficulties are encountered in obtaining representative standard size test cylinders of very light weight cellular concrete in which the air cells are produced by chemical reaction. These difficulties were not foreseen during the preparation of the proposed revised standard. Apparently the determination of f_c' and E_c for these low density autoclaved cellular concretes will have to be made by special procedures involving special test specimens.

No difficulty is foreseen in the determination of f_c' using a cubical specimen if a proper correction factor is applied to relate the strength to that obtained by a 6 x 12-in. cylinder. The factors 0.98 and 0.92 suggested by Mr. Freeman for two mix densities seem high, though it is possible that the factor would be influenced by the low strength of the concrete.

A beam deflection test can be used to determine E_cI; however, it is difficult to separate the two factors because the resisting section changes with increasing

load, the full section being effective until cracking of the tensile concrete begins. It is entirely possible to develop a safe, if not exact, procedure for determining E_c and therefore n by a deflection test.

Methods for the determination of f_c' and n for this class of concretes should be included in a future revision of the standard.

In discussing Section 401(a), Mr. Freeman attributes increased bond strength to anticorrosion treatment of the steel and to the autoclave curing, as demonstrated by pull-out tests of treated and untreated rods. When these results are applied to anticorrosion-treated welded wire mesh, a part of the apparent increase in bond strength is due to anchorage furnished by the cross wires of the mesh. It is not deemed necessary to refer to Section 101(c) which is obviously applicable. To do so would require a similar reference in a number of other sections.

In this standard it is believed that the deflection requirements of Section 301 adequately limit both elastic and plastic deflections. It may be that this is not true for all types of concrete, especially for lightweight aggregate concretes and cellular concretes. The problem is complicated because plastic action is modified by the arrangement of reinforcement. For standard, mass-produced precast members, acceptable performance of the member in service requires consideration of plastic action by the manufacturer. If this proves to be insufficient limitation, then a revision of Section 301 would be indicated.

It is considered that end-bearing requirements are satisfactorily covered by the Building Code (ACI 318-56) Section 305(a) and by test performance.

"FACTORY" CONTROL

In Section 405 the reference to "factory controlled conditions" is at least partly defined as requiring that the concrete be vibrated or forced into place under pressure. Further, in Section 402, where factory controlled concrete is mentioned, it is required that positive and rigid devices be used to insure exact location of steel. It is difficult to define "factory methods" and "factory control" very precisely except at great length. The committee's intent is that factory control, as applied to precast concrete members, includes: (a) the use of accurately dimensioned forms; (b) accurately made reinforcement assemblies; (c) positive and accurate positioning of reinforcement in the forms; (d) rigid control of concrete quality and concrete placement; (e) well controlled curing conditions; and (f) adequate storage facilities. Whether these conditions are satisfied depends largely on the competence and integrity of the factory manager. The determination of whether or not a particular operation constitutes factory operation should ultimately lie with the technical consultant of the authority having jurisdiction over the acceptance of the product. Any precasting operation satisfying the above criteria may be considered a factory operation, wherever its location. The committee will be asked to consider the possibility of defining the terms in question more fully.

TOPPING

The term "topping" as used in Section 403 is intended to apply to placement over a continuous concrete surface. Such surfaces are often not made rough for bonding and hence the limit of 0.02 f_c' on the shear stress. The allowance of 0.03 f_c' in Section 408(b) for shear between floor slab and joist flange may logically be questioned if the joist flange is not keyed or otherwise roughened to develop shear resistance. The allowable stress of $0.03f_c'$ has been used successfully in joist and slab construction for a number of years. Refer to "Concrete Slabs and Precast Joists as Monolithic T-Beams," by F. N. Menefee, *Concrete*, 1934.

Although it is not stated explicitly, it is intended in Section 408(b) that the contact perimeter between slab and joist be used in calculating shear stress.

The writer agrees that a bibliography referring to background material would be most helpful. However, in the work of preparing a standard, the pressures are such that the primary effort is directed toward the standard and the supporting data are seldom organized in presentable form. Some tests have been made to support the recommendations of Section 801 relative to lateral transfer of load by grouted joints.

Discussion of a paper by Jacob Feld:

Failures of Concrete Structures*

By EDWARD A. ABDUN-NUR, LOUIS BALOG, R. M. GENSERT, J. J. POLIVKA, E. M. RENSAA, PAUL ROGERS, HENRY H. WERNER, and AUTHOR

By EDWARD A. ABDUN-NUR†

The author has come through again with one of his inimitable papers that are always so challenging, thought provoking, and most interesting. He is to be complimented for sifting through the large mass of literature and selecting very appropriate examples of various types of failures. His classifications of the failures into 11 groups adds realism and makes a more readable and informative piece of work.

The writer agrees completely with the author's first paragraph on "supervision omission." Proper inspection is the key, and the most important factor in reducing failures to a minimum. Every engineer knows that the best designs and specifications can be ruined by improper, careless, or incompetent inspection. In these days of shortage of technical personnel, a very important key to proper inspection is training. There is hardly a job that can be properly controlled without some degree of training and indoctrination of inspectors. Another important factor in obtaining satisfactory finished structures, is the backing of the inspectors by management. Without such backing, the inspector can get nowhere with his work no matter how competent he may be.

The author's comparison between making good bread and good concrete is certainly valid. However, on the larger projects some measures such as "self-functioning" and automatic features can be incorporated into the requirements to reduce the vulnerability and hazards due to rush work. Even on the smaller jobs some of these features can be used to advantage, although not to the extent possible on the larger work.

By LOUIS BALOG‡

Structural failures direct attention to lack of knowledge or to disregard for available knowledge. Investigation of the causes of failures based on complete data may lead to improved designs. Progressive developments in making concrete and in placing concrete and steel resulted in the condition that failures occurred more and more due to inadequate design of the structure or the forms.

*ACI Journal, Dec. 1957, *Proc.* V. 54, p. 449. Disc. 54-25 is a part of copyrighted Journal of the American Concrete Institute, V. 29, No. 12, June 1958, *Proceedings* V. 54.
†Member American Concrete Institute, Consulting Engineer, Denver, Colo.
‡Consulting Engineer, Binghamton, N. Y.

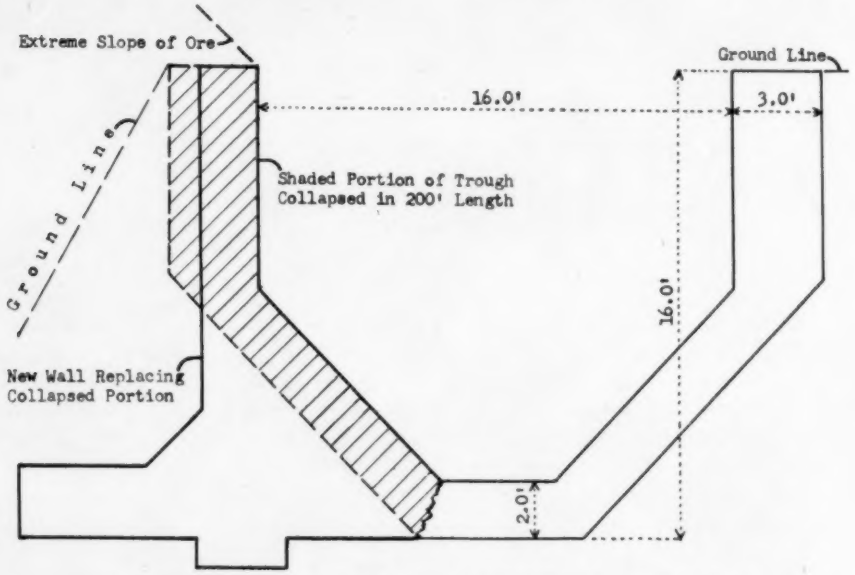


Fig. A—Storage yard ore trough collapse in 200-ft length in 1924

Structures like that shown in Fig. A failed during the 1920's due to any one of the classifications of failures by the author. Unlike such failures, that indicated in Fig. B deserves more than historical interest even at this time.

THREE-HINGED FRAMES

Fig. B shows the plan, concrete dimensions, and the reinforcement of the 34 three-hinged frames of a 620 x 96-ft structure built in Austria in 1938. After the construction of Part I, the second half was built. The 17 frames of this Part II collapsed immediately upon the removal of the forms.

The specified yield strength of the reinforcement was 56,890 psi at an elongation of 0.4 percent; the breaking elongation was 10 percent. The specified cube strength of the concrete was 4270 psi. The allowed stresses were 1420 psi (concrete) and 31,290 psi (steel). These frames, spaced 18.8 ft, carried a ribbed concrete roof slab; the ratio of the dead load to live load was about 4:1. In the uncracked condition of the concrete the computed deflection of the crown hinge was 1/550 of the span; with the concrete cracked at the tension side this deflection increased to 1/190 of the span, or to 5.65 in. The deflection of the standing frames varied from 3.28 in. to 6.18 in. The forms were not cambered.

The bearing hinges, containing ten intersecting bars, spirally tied, were precast into about 8 x 9-in. blocks. These were set into the footings, extending about 20 in. into the columns. A 2 in. thick felt layer covered with tar paper was placed around the intersecting bars at the top of the footings

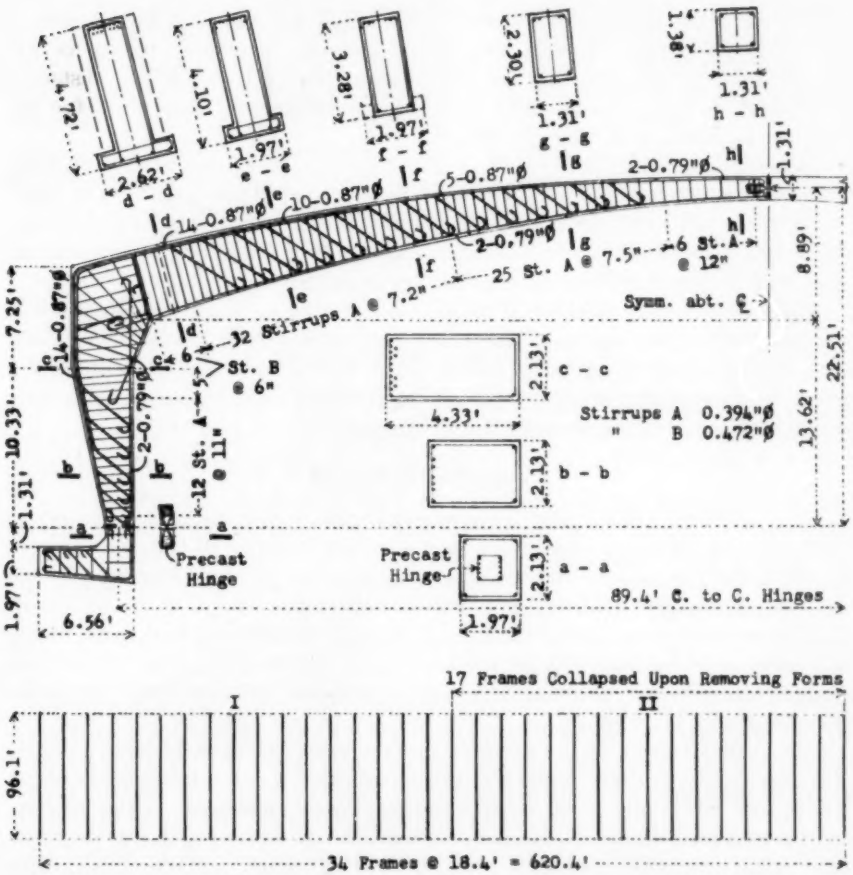


Fig. B—Collapse of three-hinged frames in 1938

to assure hinge action. After concrete was carefully placed around these hinge blocks, the columns were concreted in 20-ft heights. The wet concrete compacted the water-saturated felt completely in the 17 frames which stood up; however, ½ to ¾-in. felt gap remained between the footings and the column bases of tne 17 frames which collapsed. The only difference between the standing and the collapsed frames was that the columns of the standing frames rested on the footings.

All 17 frames failed slightly above the bearing hinges. The bond between the precast hinge blocks and the column concrete was impaired as the wet concrete compacted the felt joints, the concrete and the two stirrups encasing the precast hinge blocks were too weak to take the shear, both stirrups yielded, and the 26 in. square concrete section broke.

The most significant fact concerning these failures was that the bearing of the columns on the footings established enough continuity to assure the stability of the standing frames. Continuous construction at the locations of these three hinges would have resulted in a satisfactory structure from the materials contained in these frames.

Moment redistribution occurs in the properly reinforced statically indeterminate concrete structure as long as shear failure is prevented. The bearing of the columns on the footings made the standing frames twice redundant. Even this kind of continuity was sufficient to effect stability of these frames, also because the large bearing pressures reduced the shear.

This example indicates disadvantages of introducing hinges into excessively flexible single-span frames. Continuity reduces not only the deflections but also the shear at the column bases. The restraining moment reduces the shear because of the tapered form of the columns and the bearing pressures reduce the shear in the vicinity of the bearing.

TWO-HINGED FRAME

The failure of a 70-ft span two-hinged frame at Coronado (California) in 1954, mentioned by the author, is an instructive example of the role of knees in two-hinged frames. Only the knees uniting the two base-hinged columns and the girder give stability to a two-hinged frame. Lack of continuity in the knees make such frames collapsible due to vertical and horizontal loads. The steel must be bent around the knees on a radius defined by the allowed bearing pressure and it must be extended into the columns and the girder. This is always practicable.

The rebuilt frames of the same concrete dimensions have 12.48 sq in. total steel area in both the column and the girder sections of the knee, compared with 31.84 sq in. and 15.60 sq in. total steel areas in the respective sections of the collapsed frame. This latter frame, 20 in. wide, could not carry its own weight because the steel was cut in the knees. Efficient and safe design demands continuous steel in the knees. Overload, dynamic load, or chemical action may impair the continuity of bond spliced knee steel and bring about the collapse of two-hinged frames.

HINGELESS FRAME

Failure of a single span hingeless frame occurred because horizontal load moved the vertically loaded column top outward by 1/117 of the span of the frame. At this eccentricity of the load the column, with width-length ratio of 1:32.5, buckled.

The conditions which make any type of single span frame unstable, including such effects of the deformations, should always be ascertained.

CONTINUOUS FRAMES

Frame stability increases with increasing number of spans. Continuous frames can become more rigid longitudinally than desirable from the point

of view of temperature and shrinkage stresses. The longitudinal movements of the girders will occur without excessive stresses in the six-span, 400 ft long frames discussed by the author, if the center column is made rigid and the others, proceeding symmetrically outward, are made successively more flexible or movable. In this manner the deformations occur in two equal symmetrical lengths. The safety of structures of the proportions of these girders can be definitely and economically assured for the conventional types of loadings.

Less is known, however, concerning the design of frames in which the depth-length ratios of the members are large. Photoelastic and other model methods can help in the safe and economical design of such structures, many of which exhibit severe cracking.

SHELLS AND FOLDED PLATES

It is a significant fact that while extensive failures of girders, the oldest type of reinforced concrete structures, occurred during recent years, failures of the more novel roof shells and folded plates have been rarely reported.[50]

Large shell structures require the proper consideration of their construction accuracy and of the effect of the deformations on their stability. Increasing knowledge diminished continuously the value of buckling parameter k in the expression of the critical load on spherical shells:

$$p_{cr} = k\, E\, (t^2/R^2)$$

Its first value, $k = 2/\sqrt{3\,(1-\nu^2)}$, was derived by Robert Zoelly in 1915; for $\nu = 1/3$, $k = 1.225$. Tests of Zeiss network domes during the early 1920's revealed that a $k = 1.225$ buckling parameter was unsafe. Nevertheless, the correctness of this value was proved repeatedly by various authors until 1939 when von Karman and Tsien arrived at $k = 0.365$.

Experimental investigations of spherical shell steel models published by Köppel and Jungbluth in 1953, and by Köppel and Roos in 1956, showed that the value of coefficient k can be many times smaller than 0.365 as a function of the h/d and R/t ratios and of the support condition of the shell.

Likewise, much smaller buckling loads than those resulting from previous coefficients were indicated by the reinforced concrete spheroidal shell model tests of Torroja published in 1950. Csonka[48] analyzed these tests and established a buckling parameter value of $k = 0.069$. He found further that the behavior of the spheroidal shell shown in Fig. C defined a parameter value of $k = 0.062$; therefore, he recommended that the critical load on spheroidal shells be computed by the formula

$$p_{cr} = 0.06\, E\, (t^2/R_1\, R_2)$$

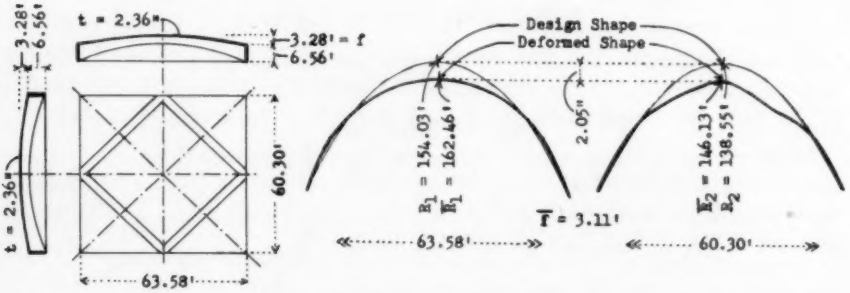
Because of the varying and low values found for the buckling parameter k, the safety factor of large shell structures should always be ascertained in design.

Notation

k = buckling coefficient
E = modulus of elasticity
t = shell thickness
R, R_1, R_2 = radii of curvature of shell
h/d = rise to span ratio of spherical shell
ν = Poisson's ratio

FORMWORK FAILURES

Collapsed formwork seldom reveals the real cause for its condition, the author observed. Reported failures of erection bents, pile bents, and formwork bents, however, often permitted the conclusion that ineffective bracing of the columns made them act as if hinged top and bottom. The lateral



(a) Design Dimensions of Shell (b) Change of Design Shape of Center Sections of Shell

Fig. C—Deformations of flat spheroidal shell in 1954. Continuity of concrete in center portion of shell was interrupted by 625 openings, 6.3 in. in diameter, spaced about 14 in. on centers both ways, and subsequently filled with glass disks. Irregular deformation of the shell began 3 years after construction

deflections of such bents create transverse loads on the structure they support. The magnitude of these loads depends on the transverse rigidity and the vertical loading of the supported structure. When the critical magnitude of the vertical load is reached, collapse occurs.[49]

Generally, the majority of formwork failures are caused by development of hinges at the splices of ineffectively braced columns.

INHERENTLY UNSAFE STRUCTURES

The failure of only one member caused the collapse of pin-connected trusses. Nevertheless, insistence on their use prevailed because of fear of secondary stresses which never caused the failure of rigid-jointed structures. Use of simple spans instead of the safer continuous spans is another example of the same attitude. The use of primitively assembled wooden roof trusses instead of shell structures, despite failure of the former and the great carrying capacity, fire safety, and economy of the latter, is a further historical example

for the insistence on unsafe construction. More recently, it became evident that brittleness of materials is an inherent characteristic which should always receive consideration in design.

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By R. M. GENSERT*

This discussion refers in particular to the mention of the erection of the eight-tier lift-slab structure in Cleveland (last paragraph, page 462). Contrary to what is stated and implied in that article, there was no failure of the concrete structure, and the erection difficulties encountered had no relationship whatever to the engineering analysis of stresses and strains. After a careful review, the authorities approved the completion of the building as designed.

Mr. Feld might try to excuse or justify his unfortunate remarks about this incident in Cleveland by referring to the fact that it was given under the subhead of "erection failures." However, as is well known, there may be erection failures by reason of fault in the original design of a structure as well as by reason of fault in the actual erection procedures. His article, if intended to confine the fault to erecting procedures, was careless and indefinite in not making such intention clear. The article may readily be read to suggest or imply that the "erection failure" was due to original design of the structure, which is not true.

The author's bibliography did not provide authority for his statements, but on the contrary, found no fault or made no criticism of the original design.

Erection of a lift-slab structure depends upon procedures that are outlined in the lift-slab manual and upon usual and accepted erection procedures. As designing engineers, we did not have supervision of this patented erection procedure.

The article stated that emergency action stopped further movement of the structure. This statement is erroneous since the displacement stopped hours before cables were tied to lamp posts, wire fences, and a few trucks. It is hard to imagine that there ever was a true failure since the structure assumed its intended frame action after the slack between collar and column was taken up.

*Member American Concrete Institute, Gensert, Williams and Associates, Cleveland, Ohio.

Although the article indicates that straightening operations were carried out in violation of the building commissioner's orders, actually he ordered the straightening to proceed. This fact was published in the local newspapers June 13, 1956. The structure was straightened by a much more elaborate process than the author implies. The operation was described in full detail in references in the author's bibliography.

The author's conclusions regarding erection of the west tower of the structure are further challenged on the basis of the later successful erection of the east tower which did not displace at all, and on the basis of the present use of the entire building, all of which have vindicated the engineering design of the structure. Since litigation to determine liability (that does not involve the engineers) is still pending in the courts, it would be inadvisable at this time to comment further on the erection causes of the building displacement; this is yet for the court to decide.

By J. J. POLIVKA*

The author's survey of concrete failures is of great value not only for safety but also for further progress and development of structures in reinforced and prestressed concrete. Any past failure, no matter how important it was, should be reported and discussed, without bias, thus contributing to greater cautiousness of the designer and the builder. Reinforced, precast, and prestressed concrete are excellent structural materials, competitive with timber and steel if properly designed; however, they require greater attention not only by the designer, but also by the builder and his subcontractors. The writer believes that special attention should be given to the following causes of damages, failures, and necessary repairs:

1. Concrete of the structure should have the specified properties. It may happen that a concrete mix complies with specifications, as proved with sample tests; however, its properties in the structure or in precast structural members are considerably weakened due to inadequate handling, curing, etc. on the construction site, lack of protection against atmospheric changes, restricted time of hardening, and other omissions.

2. High strength steel wires, especially used for pretensioning, are usually stored in seller's warehouses, and protected against rusting by surface greasing which must be reliably removed, because, if not, the basic property of this reinforcing type—the bond—is partly or completely lost.† In extreme cases the bearing capacity of the structure vanishes.

3. It is cheaper to manufacture pretensioned concrete members in a series using wires with excessive length with one pretensioning equipment. Normally, such manufacture is performed in plants protected against weather and temperature changes. For example, if 500 ft long thin steel wires are pretensioned for a number of structural members, a temperature change of 30 F causes expansion or contraction of approximately 1¼ in., which in

*Member American Concrete Institute. Lecturer, Stanford University, Consulting Engineer, Berkeley, Calif.
†See "Reader Comment," *Engineering News-Record*, July 9, 1953.

individual shorter members completely eliminates necessary bond during setting and hardening process of concrete, since there is a continual sliding of the embedded wires. This dangerous condition can be observed when wires of individual members are cut. When the concrete set with rising temperature, the wires are visibly retracted, and such members should be eliminated. Some engineering firms specializing in pretensioned concrete specify—if there is no possibility to manufacture the members in protected rooms—special skeletons for out-door pretensioning in heavy timber with adequate protection against changes in temperature, humidity, etc.

4. Connections between various precast members should be carefully designed, and well executed. If the holes provided for interconnecting reinforcement in a member, are close to the surface, the bond required for monolithic character and properties of the structure (continuity) is not secured; this condition may cause considerable increase of stresses and endanger the completed structure. If such connections are used in parts of the building where they are necessary for the required safety, and temporary supports must be installed until specified strength of the concrete is attained, the earlier removal of such supports (e.g., scaffold of a roof structure) is dangerous.

5. It is surprising that in some cases after failure it was found that the placement of reinforcement (mild steel) was not correct and not in accordance with details of the designer. In such cases the reinforcement is bent *and placed* by the sellers who often are not acquainted with the function of steel in concrete and misplace some bars which seriously contribute to the failure.

6. Most important is steady supervision by experienced engineer, especially during critical stages of the construction.

By E. M. RENSAA*

The writer read this paper with great interest. It is a useful reminder that engineers are in a special position because so much of their work is dealing with the laws of nature. It is often possible to break laws made by society and get away with it, but breaking the laws of nature to any appreciable extent brings swift and often serious punishment.

During 33 years of professional work, the writer too has seen some failures of reinforced concrete structures and even more cases where failure was averted by eleventh hour corrections.

BASIC ERRORS IN DESIGN

It may be true that design deficiency is a rare occurrence insofar as direct errors of calculation are concerned. However, every design must be based on certain assumptions of loading and flexural conditions, and herein lies a great possibility of making errors. Furthermore, design has to comply with certain code provisions, and if these are followed, the structure is then supposed to be safe. Many designers with a limited understanding of basic facts believe that those who wrote the code knew most of what was worth

*Member American Concrete Institute, Rensaa and Minsos, Edmonton, Alberta, Canada.

knowing about the subject. Codes are therefore likely to be followed blindly, also where they really do not apply. Making correct basic assumptions is therefore of primary importance, and often difficult.

A serious deficiency with most reinforced concrete codes is that they contain only minimum requirements, without giving the limits of their applicability. Provisions which will give safe designs for ordinary members may be quite unsafe to use for large and complicated ones. Design formulas for shearing stresses and bond stresses, for instance, are based on the assumption that concrete cannot resist any tension on transverse cross sections taken a short distance apart, and that is not generally true in practice. However, the formulas are very useful as a means of obtaining *relative* stress values, as long as they are used for comparable conditions. Safe allowable relative stresses obtained by testing similar reinforced concrete members may then be used for design of members. Great errors may be made if there is no proper similarity between tests and design. For instance, the allowable shearing stresses are based on tests of simply supported beams where there are both shear and bending. We cannot safely assume that shearing stresses of the same magnitude, found by the same formula, would be safe if there is no bending but only shear. That condition will apply at points of contraflexure. The stress condition at this point has been discussed in a recent paper by the writer.[51] There can be little doubt that a better understanding of the difference between bending-shear and no bending-shear conditions would have prevented the failures of the air force warehouses.

The fact that some design assumptions only apply to specific conditions is often overlooked. It is most often assumed that concrete has no tensile strength, and even sometimes assumed that it cannot take any shear either. None of these assumptions are generally correct, and concrete would be useless as a construction material if such were the case. These assumptions are, however, in many cases useful and necessary for design purposes, but they should not be used as a basis for theories of general applicability. We know, for instance, that we cannot calculate actual deflection of a reinforced concrete beam by assuming the concrete to have no tensile strength. Even so, it still happens that attempts are made to formulate general theories on basis of design assumptions.

It is not always the engineers with little theoretical knowledge that walk into such a trap. One very special case of this kind of error may be of interest. The very able German engineer, the late Franz Dischinger, published in 1942, a new formula for bond stress, in a German handbook for civil engineers.[52] This formula gave much less bond stress than obtained by the previously accepted theory. The mathematical development of the formula was based on the usual German assumption that concrete can take no shearing stress on the complete length covered by the shear diagram, when the maximum allowable unit shearing stress at the support for concrete with no shear reinforcing is exceeded. The equally noted reinforced concrete specialist, Emil Mörsch, pointed out that the values obtained by the Dischinger formula,

did not agree with test results, nor with his own theory, which was based on truss analogy. A rather extensive discussion ensured, and in the end ten of the most prominent German professors sided with Dr. Dischinger.[53]

No explanation by anyone was given as to why the Dischinger theory did not agree with actual tests. There was nothing wrong with the mathematical development of the formula, but what all the discussors seemed to miss was the fact that it was based on the incorrect assumption that concrete can take no shear. In the second edition of the same handbook, the Dischinger bond theory has been omitted. When test results and experience do not agree with a theory, we may be sure that it is the theory that is wrong.

Too little space, in the writer's opinion, is generally given in textbooks to discussion of basic facts, and formulas are developed and presented as if they may be relied on to give exact values, regardless of load and stress conditions.

SUPERVISION ERRORS

Any engineer who has had some experience on supervision, will appreciate the difficulty of this work. It is really surprising what strange interpretations of fairly clear drawings may be made by men on the job. If the inspector is inexperienced or lax, the reinforcing may be placed quite differently from what was intended. The writer has seen a picture taken on a defense project where the four column bars were placed in a bundle in the center of the column form, and the concrete then placed. Some years ago, when a building was demolished, the reinforcing in a beam was found to have been placed in the form bundled together, and even with the shipping tag left on. The writer once happened to inspect a dam project containing a fairly large spillway culvert. The resident engineer together with a second-year engineering student had decided that the designer had made a mistake, and had placed the reinforcing on the wrong sides in the wing retaining walls, and in the intake culvert. They changed the position of reinforcing in the retaining walls to the inside, and in the buried culvert to the outside, "to take care of the water pressure." The footing slabs had been cast when the writer arrived, so it took some work to make corrections.

FROST PROTECTION DEFICIENCY

A few years ago, a rather large reinforced concrete building was designed by the writer's firm, and successfully completed under our firm's supervision. The owner had acquired right of property to the drawings, and proceeded to erect a similar building on fairly wet clay ground. Column footings were cast in the fall, and left more or less without frost protection. Work on the superstructure continued through the winter, but when the ground thawed next spring, some parts of the building settled up to 3 in. Frost had penetrated under the footings, and lifted some of them before the superstructure had been formed and concreted.

The writer thinks that it would be a great service to the engineering profession if an engineer with such a long and extensive experience as the author's, would write a book describing errors and failures of concrete structures. The author, in his paper has, in any case, given a valuable contribution within the limited space utilized. It is very useful for anybody, and especially for younger engineers, to know where trouble has been encountered, and where it may be expected to happen again.

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By PAUL ROGERS*

The author should be commended for his interesting and thorough discussion of failures of concrete structures. As he pointed out, most of such failures escape publicity unless there is a loss of life or heavy material damage.

The writer would like to describe, briefly, a few recent failures on which he was called as a consultant. The first case involved a new public school gymnasium. The roof framing was a reinforced concrete slab and beam system. The beams had 54-ft spans framing into deep, narrow spandrel beams. Although this structure was designed by a reputable structural engineer, an error entered into the computations so that less than one-third of the necessary reinforcement was provided. The error was discovered after the concrete was cast, but the formwork was still in place.

The writer recommended that Stressteel bars of adequate strength be applied along the stem of the beams. Holes were drilled through the spandrel girders and heavy steel plates resisted the anchorage pressures on the outside. The bars were tensioned to 100,000 psi. The final result was a partial prestressing system which allowed the mild steel reinforcement to be stressed up to 20,000 psi.

Another case involved a canopy for a railroad siding, adjacent to a warehouse. The basic system consisted of 16-ft cantilever frames. The structure was properly designed, and the bars were correctly detailed and fabricated. The engineer insisted that he be notified before the placing of concrete so that he could inspect the cantilever reinforcing. He was not notified, and later the cantilevers showed signs of definite tension failure, all of it at the end of the bars bent out from the column. After removing one of the cantilevers and finding the bars at the bottom instead of the top, it was revealed that the contractor had entrusted his superintendent to see that the bars were placed properly. The superintendent asked the foreman to supervise it, and

*Member American Concrete Institute, Consulting Engineer, Chicago, Ill.

he, in turn, asked the iron setter. The iron setter decided that the engineer's drawing was all wrong, and placed the cantilever bars on the bottom. The writer suggested a remedy similar to the case above.

Still another case involved a new school. Although it was designed by a very competent architect who always employs a structural engineer for his larger projects, this structural system was developed by the architectural designers of the firm. It consisted of concrete block bearing walls, precast and prestressed joists, precast columns, and some cast-in-place beams and cantilever canopies. The members were well designed for flexure and shear, but the whole system was loose and unstable. Serious cracks and movements took place almost immediately after construction, and the writer had to recommend rather extensive remedies in order to salvage the structure.

That strict field inspection is a "must" is manifested by the next case, which involved an expansion joint. This consisted of a heavily reinforced bracket upon which a long, heavy girder rested. After a rather sudden drop in temperature, half of this bracket literally sheared off. The writer insisted on the complete removal of this cracked portion, which revealed that the separating beam seat was omitted, and almost full bond had developed between the bracket and the beam. The heavy cantilever bent bars were all grouped and "wired" together at about 10 in. from the face of the bracket. The heavy horizontal closed ties slid down *below* the bracket, and the portion which cracked off was unreinforced concrete, incapable of resisting the tension and shear forces. In this case, the writer had to suggest a complex structural steel system to restore the capacity of the brackets.

The last case concerns the collapse of the formwork for a large prestressed concrete water tank, with the loss of one life, and injuries to another worker. The writer, while serving as a member of the coroner's jury, discovered that an engineering firm, which specializes in assisting smaller municipalities, had placed its engineering seal on drawings prepared by the contracting firm. Apart from the fact that this may be considered an unlawful procedure, it resulted in a split responsibility, followed by careless supervision, improper bracing, inadequate nailing, and, finally, collapse.

The writer believes that engineers would do the public a great service by describing structural failures which come to their attention.

By HENRY H. WERNER

The author has presented an excellent survey of this delicate subject. His classifications, analysis, warnings, and advice are clear and well taken. After all, we learn a lot by failures, so much so, that testing to failure is a basic procedure used in our laboratories. Yet, no testing machine can reproduce all the variants which affect field work. Many failures cited in the early history of concrete construction, and publication of these failures, certainly did drive home a lesson or we would not have made such good progress in this field.

*Member American Concrete Institute, Consulting Engineer, Long Island City, N. Y.

We now are entering a period when new materials, methods, shapes, etc., enter our construction field, and again a greater number of failures is being reported. The more and the sooner we learn from these, the better for all concerned. Unfortunately, we are only talking about the failures that "made the news." It is this writer's opinion that the great number of "small" failures, occurring continuously, add up to more loss than those that are publicized. The lessons to be learned from these innumerable mishaps are not studied and driven home, much to the loss of the industry.

The first step should be a systematic collection of data on failures, i.e., disagreement with expected conditions as defined by the author. Now, this is quite a formidable and delicate task which could only be tackled by a neutral agency of greatest standing, having the support and confidence of the whole industry. Such agency would constantly contact all levels of concrete construction to reach the maximum number of cases and collect data. It should not take sides, publish names or cause expense or annoyance to anyone involved.

The next step would be to analyze and classify the material along the lines so well presented by the author. Finally, this survey should be published periodically to reach all concerned. This may sound like an impossible undertaking, requiring a broad campaign to gain confidence and great perseverance to follow up. It will be hard to overcome the reluctance of anyone involved in a failure to divulge information to such a data-collecting agency. Yet, the results should be worth the effort.

From my own experience I can say, that certain errors and failures could be avoided if instructions that came with a number of new construction materials and methods would be clearer, even to the point of stressing a dangerous pitfall. Apparently some manufacturers prefer to camouflage a potential danger rather than expose it for the users' benefit. It also is amazing that so many contractors and foremen have not yet understood the basic points to be observed when substituting reinforcing bars. Judging from experience, I would say that many of the so-called small failures and subsequently a number of big ones, could be avoided if the lessons to be learned from them could be widely collected and widely circulated.

AUTHOR'S CLOSURE

The purpose of this paper was to bring attention to unwanted incidents that plague the construction industry, in all types of work. The attention which the paper received indicates that the seriousness of the problem is individually recognized. Pooling of such feeling to the extent that as far as possible, rapid exchange of experiences will be provided through such media as the ACI JOURNAL can only result in the reduction of the number of repetitive failure types. It is rather discouraging that since this paper was originally written, some 2 years ago, the reported new incidents have been so many, enough to write another paper. Hopefully, the real reason for this apparently large number of reported incidents may be the loss of reluctance

to report unfavorable action. During the past few days (March 1958) the press notes that for the first time in many years, the rate of traffic accidents is actually decreasing and credits the continual publicity of accidents as the major contributing factor to this decline. We all hope for a similar result from the publication of all construction accidents and failures.

The men who discussed the paper are to be commended for writing so frankly of their own experiences. The almost unanimously favorable comments in the correspondence received are appreciated.

Mr. Balog's long experience in special types of structures is evident in his discussion pointing out that every small detail must be investigated and designed to be consistent with the assumptions of the design. The structure is not advised of the designer's assumptions, so the assumptions must conform to a design that is buildable. It might be good for lecturers in design courses to ask the question: "Is your design buildable?" Not in the sense of assembling an intricate mechanism, but in putting together a successive set of construction procedures, each with its necessary tolerances to compensate for less than perfect materials and possibly not the best of trained personnel.

The example of the hingeless frame failure, could have been avoided if the necessity for lateral bracing were recognized for all compression members, whether horizontal portions of rigid frames or vertical columns. Lateral deflection of formwork is undoubtedly the basic cause of such failures as Mr. Balog recognizes.

The writer agrees completely with Mr. Werner that it is not only an advantage but a duty of the profession to leave a clear record of instruction of *what not to do* if successful designs are to be prepared.

Mr. Rogers' experiences in correcting errors discovered in partially completed structures closely parallels that of the writer. A long span solid slab which showed too much deflection was corrected by introducing a cap slab, 4 in. thick, after shoring the deflected span and wedging it upward, and adding high strength steel mesh welded to studs shot into the concrete over part of the deflected span and the complete (shorter) adjacent span. A considerable portion of the unwanted deflection was eliminated. The writer also had the experience of ordering a cantilever canopy removed and rebuilt with the tension rods placed on the top of the slab where the design drawings had properly shown them to be placed. In a large underground garage attached to an apartment house, the roof girders were shown to rest on brackets of the main building columns on sliding seats. In several locations, indiscriminately placed, the seat had not been smoothed and during the next winter, brackets cracked and pulled away. Repairs were made at a cost many times the small labor cost saved in the finishing.

Mr. Gensert's discussion, points to the fact that control of construction, especially of unusual designs, is an important function of the designers. The righting of the deflected structure described is a feat which can teach many lessons. It is hoped that Mr. Gensert will describe this fully for the benefit of the profession.

In reply to Mr. Rensaa, the writer believes that articles of this type are better than a book on the subject. The data can be presented more rapidly and to a larger reading audience. A book, like a code, becomes the last word on the subject; articles can follow each other as data becomes available and incorporate the experience of the several authors. It is strange how the repetition of code requirements in a few years becomes "recognized practice" even though the original material may have been a stop-gap to fill out the outline of the code contents, awaiting more complete investigation. The shear requirements mentioned by Mr. Rensaa are a case in point.

Mr. Abdun-Nur seems to have had the same trouble with inspection as has been found by the writer. Expediency on the job often cuts into the factor of safety, usually not to the point of failure, but the margin is often very close.

Mr. Polivka repeats some of the warnings heard in recent meetings discussing prestressed concrete. To these, the writer would add the extreme hazard of frost cracking during winter grouting and the danger of expecting the concrete work in the usual congestion of bars and sheathes to closely follow the desires of the designer. The salesman may argue that prestressed concrete being always in compression cannot crack, but somehow the results contradict such an evident statement. Perhaps it may also be pertinent to quote the order reported to have been issued by General Goethals to his staff at the Panama Canal: "Make your instructions so clear that the meanest intellect can have no reason for miscomprehension."

Discussion of a paper by Tung Au:

Ultimate Strength Design Charts for Columns Controlled by Tension*

By JACK MOYSE and AUTHOR

By JACK MOYSE†

I found it difficult to understand the advantage gained by use of the charts for rectangular columns with symmetrical reinforcement as the computation required to use the charts is even more than that required for a direct solution.

For example, the formulas listed below will give a direct solution without recourse to charts.

$$A_{st} = \frac{P_u}{f_y\, t_s}\left[\frac{P_u}{0.85b\, f_c'} + (2e' - t)\right] \quad \ldots\ldots\ldots\ldots\ldots\ldots (10)$$

$$b = \frac{P_u^2}{0.85\, f_c' \left[A_{st}\, f_y\, t_s - P_u\,(2e' - t)\right]} \quad \ldots\ldots\ldots\ldots\ldots\ldots (11)$$

$$t = \frac{P_u}{P_u + A_{st}\, f_y}\left[2e' + \frac{P_u}{0.85b\, f_c'} + \frac{2A_{st}\, f_y\, d'}{P_u}\right] \quad \ldots\ldots\ldots\ldots\ldots\ldots (12)$$

Using Example 1, p. 479, with $P_u = 300$ kips, $e' = 15$ in., $f_c' = 3000$ psi, $f_y = 40{,}000$ psi, $b = 16$ in., $t = 20$ in., and $d' = 2.5$ in., $t_s = 15$ in.

$$\frac{P_u}{f_y\, t_s} = \frac{300}{40 \times 15} = \tfrac{1}{2}$$

$$\frac{P_u}{0.85b\, f_c'} = \frac{300}{0.85 \times 16 \times 3} = 7.35$$

$$(2e' - t) = 30 - 20 = 10$$

$$A_{st} = \tfrac{1}{2}\,[7.35 + 10] = 8.68 \text{ sq in.}$$

Note that the result indicated a saving of 0.28 sq in. in reinforcement over results obtained from the chart.

*ACI Journal, Dec. 1957, *Proc.* V. 54, p. 471. Disc. 54-26 is a part of copyrighted Journal of the American Concrete Institute, V. 29, No. 12, June 1958, *Proceedings* V. 54.

†Member American Concrete Institute, Bridge Engineer, Public Works Division, U. S. Operations Mission to Cambodia, Phnom Penh, Cambodia.

AUTHOR'S CLOSURE

Mr. Moyse has underlined the simplicity of the ultimate strength design method in his discussion. The point is well recognized and has been ably stressed by Ferguson[5] that for sections with symmetrical reinforcement, the solutions are so simple that design aids are hardly needed. There are others, however, who view the lengthy formulas with great distaste and may prefer design charts which have been made available. For instance, Mensch, who has contributed to the development of ultimate strength theory, remarked:[6] "It is more difficult to analyze the problem of an eccentrically loaded column by the plastic theory than by the straight line theory; but not sufficiently so as to require 70 equations of distinguished lengths and doubtful validity as done by this [ACI-ASCE] report to justify equations [for eccentrically loaded columns]." If the paper has not completely dispelled such skepticism, Mr. Moyse's discussion certainly should remove the last doubt.

In conclusion, the author thanks Mr. Moyse for the opportunity to emphasize the simplification of design by ultimate strength method.

REFERENCES

5. Ferguson, P. M., "Simplification of Design by Ultimate Strength Procedures," *Proceedings*, ASCE, V. 82, No. ST4, July 1956, Paper No. 1022.

6. Mensch, L. J., Discussion of "Report of ASCE-ACI Joint Committee on Ultimate Strength Design," *Proceedings*, ASCE, V. 82, No. ST3, May 1956, Paper No. 972, p. 34.

Discussion of a paper by Truman R. Jones, Jr., and Henson K. Stephenson:

Proportioning, Control, and Field Practice for Lightweight Concrete*

By E. L. HOWARD†

Messrs. Jones and Stephenson have covered the workability problem of lightweight concrete very well. At least, we have done and do the things they have found to be best practice. We can also concur with that section of the paper dealing with field practice.

The authors say they have had difficulty making specific gravities. We use the pycnometer method with good results. We determine the saturated surface dry condition of fines by rolling the material in a dry glass jar. When the fines are surface dry they will not cling to the glass surface.

We are sure the authors' batch proportioning methods are far too complicated. A good workable mix can be made following the procedures offered by ACI Committee 613 in "Recommended Practice for Selecting Proportions for Concrete (ACI 613-54)." This proportioning system lends itself readily to field adjustments necessitated by changes in materials, yield, etc. This design method has been used on a dozen large jobs totaling about 40,000 cu yd. Data gathered from some of these jobs have been reported in the ACI JOURNAL "Letters from Readers" and/or "Concrete Briefs" sections: (1) "Close Control of 15,000 cu yd of Haydite Concrete for a 27-Story Building," ACI JOURNAL, June 1955, *Proc.* V. 51, pp. 1061-1064; (2) "Inspection and Control of Haydite Concrete," ACI JOURNAL, Oct. 1957, *Proc.* V. 54, pp. 357-359.

*ACI JOURNAL, Dec. 1957, *Proc.* V. 54, p. 527. Disc. 54-29 is a part of copyrighted JOURNAL OF THE AMERICAN CONCRETE INSTITUTE, V. 29, No. 12, June 1958, *Proceedings* V. 54.

†Member American Concrete Institute, Chief Testing Engineer, Pacific Cement & Aggregates, Inc., San Francisco, Calif.

ACI News Letter

Vol. 29, No. 12 JOURNAL of the AMERICAN CONCRETE INSTITUTE June 1958

Concrete shields sodium reactor . . . p. 3

Contributions to the

ACI JOURNAL

Technical pages of the ACI JOURNAL offer a medium for reporting what is new and interesting in concrete research, design, construction, maintenance, or manufacturing. ACI members and JOURNAL readers are urged to share their knowledge and experience by submitting papers and reports for publication in the ACI JOURNAL. Time and changing interests demand that new contributors with new ideas take their place beside the old.

●

The Technical Activities Committee selects the papers, committee reports, discussions, and other contributions for JOURNAL publication. A volunteer corps of experts in various segments of the field assists in technical appraisal of the manuscripts. Technically speaking, acceptable contributions present original material for improvement of design, manufacturing, or construction; confirm, revise, or upset established ideas or practices; or review, digest, or arrange accumulated experience for more ready use.

Further questions raised by TAC: will the contribution fit into a balanced publication schedule? Can it be published at reasonable expense to the Institute?

●

Contributors should furnish glossy prints or inked tracings of illustrations along with manuscripts submitted in triplicate for publication. Details of the form and physical arrangement of contributions are given in the 1957 *ACI Directory* p. 14. Separate copies of this "publications policy" are available free on request to the Secretary.

Manuscripts of papers, discussions, and reports should be sent in triplicate to:

Secretary, Technical Activities Committee

AMERICAN CONCRETE INSTITUTE

P. O. BOX 4754, REDFORD STATION, DETROIT 19, MICHIGAN

ACI

News Letter Contents

June 1958

On the Cover—Sodium Reactor Experiment Plant built for the Atomic Energy Commission about 30 miles northwest of downtown Los Angeles by Atomics International, a division of North American Aviation, Inc. Dense concrete shielding surrounds the radioactive portions of nuclear power plant. Photo shows shields around the reactor and primary sodium system.

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New Standards

ADOPTION OF NEW STANDARDS of recommended practice for the concrete industry increases the current list of ACI standards to 14. Of the four standards adopted by the American Concrete Institute at its 54th annual convention and ratified by letter ballot of the Institute membership June 1, 1958, two are new, and two are revisions of existing standards. "Test Procedure to Determine Relative Bond Value of Reinforcing Bars (ACI 208-58)" was prepared by ACI Committee 208 under the chairmanship of C. P. Siess, research professor of civil engineering, University of Illinois, Urbana. "Recommended Practice for Design of Concrete Pavements (ACI 325-58)" is the work of ACI Committee 325, under the chairmanship of E. A. Finney, director, Highway Research Laboratory, Michigan State Highway Department, Lansing.

The revised standard "Specifications for Concrete Pavements and Concrete Bases (ACI 617-58)," developed by ACI Committee 617 under the chairmanship of H. F. Clemmer, materials engineer, Department of Highways, District of Columbia, Washington, D. C., supersedes the 1951 ACI standard on the same subject. "Minimum Standard Requirements for Precast Concrete Floor and Roof Units (ACI 711-58)" supersedes its 1953 counterpart. It was prepared by ACI Committee 711, whose chairman is Leo M. Legatski, professor of civil engineering, University of Michigan, Ann Arbor. The four standards will be published in their entirety in the July issue of the JOURNAL.

Determining bond value of reinforcing bars

"Test Procedure to Determine Relative Bond Value of Reinforcing Bars (ACI 208-58)," provides a uniform basis for comparison of bond qualities

of different reinforcing bars. Prof. H. J. Gilkey, theoretical and applied mechanics department, Iowa State College, Ames, long active as chairman of Committee 208, deserves recognition for his untiring efforts in the development of this standard.

Published identically as information in 1945, this test procedure had a thorough workout through its use in extensive testing programs culminating in the evolution of specifications covering minimum geometric requirements. This test method uses companion beam type specimens, cast horizontally in inverted and normal positions so that the effects of water gain and settlement of concrete are included in the evaluation. The concrete surrounding the bars is in tension as bars are ordinarily used, which makes the results more acceptable to some users. Minimum criteria for acceptance are not included since the purpose is merely to establish relative bond values for the different bars under consideration.

Eleven drawings, full JOURNAL page size, detail the test procedure and equipment. Large blueprint copies of the drawings are available (22 x 35 in.) in complete sets at $36.00 per set from ACI headquarters.

Two concrete pavement standards

"Recommended Practice for Design of Concrete Pavement (ACI 325-58)" based on practice proved successful in the United States, presents recommendations for the design of rigid concrete pavements and bases.

It offers comprehensive directions for design of rigid airport and highway pavements or bases for conditions of climate, traffic, available construction materials and equipment, and construction methods of the United States. Included are recommendations for soil foundations, selection of slab dimensions, joints, and details for reinforced or nonreinforced pavement. Design practices of all principal state and federal agencies concerned with paving have been reviewed and correlated with the limits recommended.

"Specifications for Concrete Pavements and Concrete Bases (ACI 617-58)," superseding the 1951 ACI standard on the same subject, applies to construction of portland cement concrete pavements and bases under normal conditions for both highways and airports. Standards and specifications of several other organizations are incorporated by reference.

Sources and acceptance standards for materials are given, as well as materials testing procedures and procedures for test of concrete strength to be used as a basis for proportioning of mixtures. Specifications for the materials and construction of soil foundations for concrete pavements and concrete bases are included. Provision is made for use of foundations stabilized by a cementing agent, but materials and construction of such are beyond scope of this specification.

Materials, dimensions, setting, and removal of forms are treated. Construction methods are specified for forming joints, installation of joint seal and load transfer devices, and placing of reinforcement. Concrete proportioning based on design for minimum strength is covered in detail; proportioning based on fixed cement content is allowed.

Other sections cover production of high-early-strength concrete and the handling and mixing of materials. Detailed requirements are given for placing, finishing, and curing of pavement concrete. Protection of finished pavement, opening it to traffic, and public use of thoroughfare during construction are also specified.

Precast concrete floor and roof units

"Minimum Standard Requirements for Precast Concrete Floor and Roof Units (ACI 711-58)" prepared by ACI Committee 711, under the chairmanship of Leo M. Legatski, professor of civil engineering, University of Michigan, Ann Arbor, replaces ACI 711-53.

These minimum standard requirements for single units or multiple element assemblies are meant to be used in conjunction with the ACI Building Code (ACI 318-56), which has been incorporated in many municipal building codes. The new standard on precast units covers materials; design principles; manufacture, including curing and handling; testing of completed units; installation plans; and special provisions for holes and openings in members. The design chapter treats such problems as dimensions, allowable deflection, structural concrete topping, reinforcement anchorage and location, and use of lightweight concrete.

ACI's Southern California Chapter Meets

Leon Stein seated far left at speaker's table next to Samuel Hobbs. Charles Peterson is to the right of Mr. Hobbs

The Southern California Chapter of ACI met April 24 in Los Angeles with a program planned to be of special interest to both designers and construction men, outlining research and field experience. Speakers for the evening included Leon Stein, senior structural engineer and Charles Peterson, principal structural engineer, both of the California State Division of Architecture.

Mr. Stein discussed design and research in relation to practice in prestressed concrete construction; Mr. Peterson reported on field experience and included in his remarks a discussion of such items as the effects of overall shortening and the problems of deflection of slabs.

Henry M. Layne, structural engineer, Los Angeles, is chairman of the Southern California Chapter Committee, and Samuel Hobbs serves as secretary-treasurer of the group.

ACI releases publication on concrete for radiation shielding

A compilation of papers previously published in the ACI JOURNAL on the use of concrete for shielding nuclear radiation and the determination of proportions and properties of various heavy concretes has just been released in the form of a 135-page book.

The shielding of nuclear radiation is discussed, with emphasis on factors related to concrete technology and cost. The types of radiation and mechanics of radiation shielding are presented briefly, and data regarding concrete for shielding are reviewed. Tables and curves of concrete thickness required for shielding are given for both ordinary and heavy concrete. Problems involved in the use of special concrete are discussed and an outline of procedure for designing concrete shielding is presented.

"Concrete for Radiation Shielding, Compilation No. 1" contains the following papers:

"Concrete for Radiation Shielding," by EDWIN J. CALLAN

"Absorption by Concrete of X-Rays and Gamma Rays," by B. E. FOSTER

"Properties of High-Density Concrete Made with Iron Aggregate," by HAROLD S. DAVIS, FREDERICK L. BROWNE, and HARRY C. WITTER

"Heavy Steel-Aggregate Concrete," by E. I. FIESENHEISER and B. A. WASIL

"Properties of Heavy Concrete Made with Barite Aggregates," by L. P. WITTE and J. E. BACKSTROM

"Magnetite Iron Ore Concrete for Nuclear Shielding," by J. O. HENRIE

"Proportioning of Mixes for Steel Coarse Aggregate and Limonite and Magnetite Matrix Heavy Concretes," by DAVID L. NARVER, JR.

Institute members may now order the publication from ACI headquarters at $3.00 per copy. Price to nonmembers is $4.00.

ACI Technical Committees

Committee Rosters as of May 1, 1958**

Technical Activities Committee

Organized 1947

The Technical Activities Committee is responsible, under the Board of Direction, for technical publications, convention programs, and technical committee activities (see p. 25, *1957 ACI Directory*). Terms of members expire at the annual convention of the year indicated. The President of the Institute is a member during and by reason of his tenure of office; similarly the Secretary-Treasurer of the Institute is secretary of the committee. The chairman, president, and secretary constitute the committee's executive group.

WALTER J. MCCOY (1959) Chairman
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G. E. BURNETT (1960)
E. A. FINNEY (1960)
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Standards Committee

Organized 1937

The Standards Committee supervises the promulgation of standards according to rules adopted by the Board of Direction (see p. 25, *1957 ACI Directory*).

FRANK KEREKES (1959) Chairman
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CHARLES H. SCHOLER (1959)
CHARLES S. WHITNEY (1960)

Committee 115—Research

Organized 1921

Mission—This committee was organized to affiliate the interests of research workers and agencies in the field of concrete. Its assignment is to review and correlate research in concrete and reinforced concrete and to consider research methods and objectives.

Activities—A compilation of research projects is prepared and distributed each year at the open session of the committee at the annual convention.

The 1958 compilation is divided into three parts. The first part lists 39 educational institutions that are carrying out concrete research and gives the titles of the projects that are underway. The second part provides similar information for 42 noneducational organizations in the United States and foreign countries. In the third part the projects are classified under subject headings.

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**The next complete committee roster will appear in the 1959 ACI DIRECTORY.

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Committee 116—Nomenclature

Organized 1957

Mission—Committee 116 is responsible for maintaining an up-to-date glossary of terms in the field of cement and concrete technology. The committee will recommend standard terminology for materials, conditions, and practices for which there is now a confusion of terms. Liaison will be maintained with other organizations in more specialized segments of the field to assist in the determination of generally accepted usage. As a standing mission, the committee will solve problems in terminology created by new developments for periodic additions to maintain the glossary current.

Activities—It is the intention to develop as comprehensive a glossary as practicable, limited to terms used in the field of cement and concrete. The initial report may be restricted to terms in common usage including those generally accepted and those for which usage varies. Where a confusion in usage exists, the committee function is to resolve it by recommending a rational definition of, and distinction among, terms. Subsequent revisions of the report will include solution of problems in terminology for new developments to prevent any increase in confusion of terms. Liaison with specialized organizations in the field will serve to define the scope of activity for Committee 116 and simultaneously to avoid conflict with recommendations developed by other organizations.

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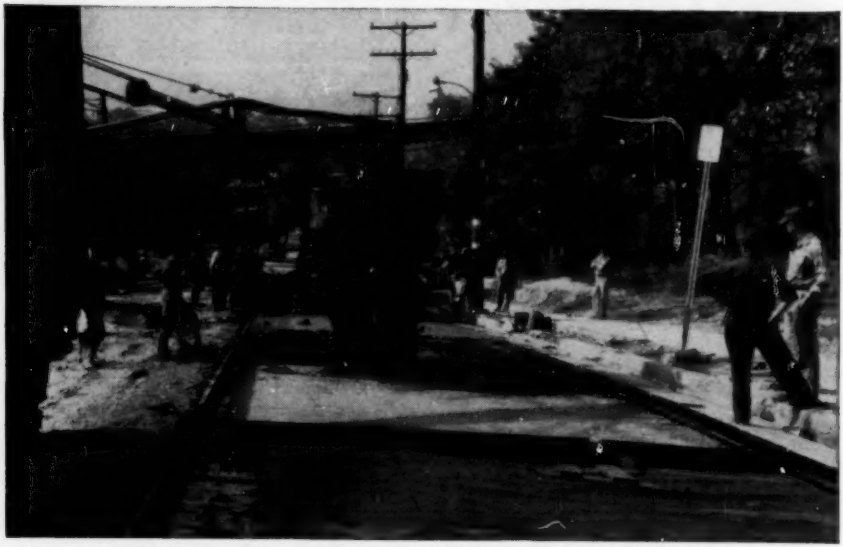

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Texas Industries, Inc.

Committee 201—Durability of Concrete in Service

Organized 1957

Mission—The committee is to review research on durability of concrete and past and present recommendations to insure satisfactorily durable concrete. It will report recommendations for materials and methods to secure concrete with maximum resistance to deterioration, to preserve concrete against deterioration, and to restore deteriorated concrete. The report may consist of suggested specifications or recommended practices to achieve the above objectives under conditions of exposure to various agencies of deterioration for common structures and pavements.

It has been suggested that the subject, durability, include resistance to abrasion, freezing and thawing, ice removal agents combined with freezing and thawing, and destructive (chemical) agents, but not erosion in hydraulic structures nor fire resistance. The report is not intended to present research results, which may be sufficient to justify a separate bibliography report. Discussion of research results leading to recommendations may precede recommendations if not available in bibliography references. The scope will be limited to material suitable for specifications or a basis for specifications; each division of the subject is to be treated individually in the most appropriate manner.

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Los Angeles Harbor Dept.

Committee 207—Properties of Mass Concrete

Organized 1930

Mission—Committee 207 is responsible for reporting developments in mass concrete construction. One main objective is to consider and report on the long-time behavior of mass concrete structures in service.

Activities—The inspection of mass concrete structures is a continuing project, with a mass of data being compiled. It is expected that these data will prove useful in preparing a report on performance of mass concrete. An attempt is being made to prepare an index on dams in which pertinent facts of construction methods, materials, exposures, performance, and other data would be catalogued.

The committee sponsored a session on the St. Lawrence seaway and power projects at the 54th annual convention in 1958.

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Cost trend is down with REINFORCED CONCRETE

Another significant example of the trend toward reinforced concrete construction is the Morton Salt Building. The architects chose reinforced concrete flat slab construction because of its lower cost, compared to other types of framing usually associated with office building as a type. As the architects point out, one of the factors in computing relative costs of construction is time; and availability of materials is an important factor when starting times are considered. They stated that the relative availability of reinforcing steel permitted the job to get under way at the earliest possible date. *Before you build*, investigate this economical, flexible, and timesaving medium of construction.

CONCRETE REINFORCING STEEL INSTITUTE
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MORTON SALT BUILDING, 110 North Wacker Drive, Chicago
Graham, Anderson, Probst and White, Architects-Engineers

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California State Dept. of Water Resources
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Office, Chief of Engineers, Dept. of the Army

Committee 208—Bond Stress

Organized 1930

Mission—This committee is assigned to the promotion of research and to the collection and interpretation of data from field and laboratory to determine proper working stresses for bond in relation to type of bar employed in reinforced concrete construction. With regard to design matters the role of Committee 208 is purely advisory.

Activities—The principal activity of the committee during the past year related to the new ACI Standard "Test Procedure to Determine Relative Bond Value of Reinforcing Bars (ACI 208-58)" which will be published in the July JOURNAL. It was approved as an ACI Standard at the 54th annual convention and ratified by letter ballot.

A subcommittee was appointed to work up a suggested program of research on bond, with special emphasis on splices, anchorages, development length, and cut-off bars, as well as the relation between bond and shear failure. Future work of the committee will depend in part on the report of this subcommittee. The committee will also focus its attention on those questions regarding bond that must be answered in connection with ultimate strength design and, eventually, with limit design.

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Laclede Steel Co.
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American Iron and Steel Institute

*Nonmember ACI.

Committee 209—Volume Changes and Plastic Flow in Concrete

Organized 1930

Mission—The assignment of Committee 209 is to review critically the available data on the factors affecting the magnitude of volume changes and plastic flow of concrete; to suggest topics of needed research, and to stimulate the preparation of papers to interpret the available information in a form useful to the designers of concrete structures.

This committee was reorganized in 1946 to consolidate the previous assignments of Committee 102, Volume Changes in Concrete, and Committee 109, Plastic Flow. Since that time the advent of prestressed concrete has introduced problems which were not anticipated in 1946, and the committee was again reorganized in 1953.

Activities—The committee sponsored a session on shrinkage of concrete at the 52nd annual convention in 1956. Five papers were published in the February 1957 JOURNAL.

Committee members are actively engaged in research in this field with the view toward publication of data useful to the structural designer. A bibliography on "creep in concrete" is also being prepared.

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C. A. WILLSON
American Iron and Steel Institute

Committee 210—Resistance to Erosion in Hydraulic Structures

Organized 1940

Mission—Reorganized in 1947, the committee was given the assignment of studying

all types of erosion occurring in hydraulic structures.

Activities—The committee report "Erosion Resistance of Concrete in Hydraulic Structures" was published in the November 1955 JOURNAL, *Proc.* V. 52, p. 259. The report is concerned mainly with the physical erosion of concrete in hydraulic structures resulting from particles carried by flowing water and from pitting resulting from cavities forming and collapsing in water flowing at high velocities. Disintegration of concrete by chemical attack is also discussed. Materials, mix proportions, and construction procedures which will make concrete more resistant to such erosion are covered.

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Swedish Cement and Concrete Institute

Committee 212—Admixtures
Organized 1943

Mission—This committee studies, assembles, and reports information on the effect of various admixtures, including air-entraining agents, on the properties of concrete. An admixture is defined as "a material other than water, aggregates, and portland cement (including air-entraining portland cement and portland blast-furnace cement) that is used as an ingredient of concrete and is added to the batch immediately before or during its mixing."

Activities—The committee's latest report, "Admixtures for Concrete," ACI JOURNAL, Oct. 1954, *Proc.* V. 51, p. 113, reported on the advantages and limitations of various types of admixtures.

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Committee 213—Properties of Lightweight Aggregates and Lightweight Aggregate Concrete
Organized 1946

Mission—This committee's assignment is to gather, correlate, and report available information on the properties of concrete made with mineral lightweight aggregates in the fields of monolithic structural concrete, precast concrete, and insulating and roof and floor fill concrete. A secondary mission of the committee is to report on design and construction practices in the field of lightweight concretes.

Activities—There are three subcommittees working on various phases of this assignment: (1) structural lightweight concrete; (2) lightweight concrete masonry; and (3) insulating concrete.

The committee also sponsored a symposium on lightweight aggregates and lightweight concrete at the 1955 ACI convention. The four reports were published in the October and November 1956 JOURNALS (*Proc.* V. 53).

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Bureau of Public Roads

Committee 214—Evaluation of Results of Strength Tests of Concrete

Organized 1946

Mission—The committee's assignment is to analyze the variations which occur in the strength of concrete, to present tools of statistics which are useful in interpretation of these variations, and to discuss measures of reliability which can be used in establishing specifications and design criteria for the strength of concrete.

Activities—This committee produced the ACI Standard "Recommended Practice for Evaluation of Compression Test Results of Field Concrete (ACI 214-57)," which was published in the July 1957 Journal, *Proc.* V. 54, p. 1.

W. A. Cordon, Chairman
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National Ready Mixed Concrete Assn.
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Charles S. Whitney
Ammann and Whitney

Committee 215—Fatigue of Concrete

Organized 1947

Mission—The assignment of this committee is threefold: (1) to review the available data on the behavior of plain and reinforced concrete under repeated loading, and to prepare a report on the present state of knowledge in the field; (2) to consider the implications of this knowledge in the design of members and structures of plain and reinforced concrete; and (3) to determine the gaps in our knowledge, and to recommend and encourage research which will provide the information to fill these gaps.

Activities—A final draft of an annotated bibliography on fatigue of concrete is in preparation with a view toward publication in the near future.

The committee also sponsored a session on fatigue of concrete at the 54th annual convention.

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Highway Research Board
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Committee 216—Fireproofing or Fire Protection of Structures

Organized 1955

Mission—This committee will study available data with a view to recommending proper ratings in concrete structures to make them more consistent and more in line with ratings given structures of other materials.

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Office, Chief of Engineers, Dept. of the Army

Committee 312—Plain and Reinforced Concrete Arches

Organized 1930

Mission—The original mission was to review and correlate all information on arch design. This mission was later modified to include the preparation of a recommended practice or design manual.

Activities—This committee has published three reports in the ACI JOURNAL on reinforced concrete arch design—correlating much that has been learned of the influence of dead and live loads, and volume changes due to shrinkage, plastic flow, and temperature change. The most recent report, May 1951 JOURNAL, presented a general design method, and a final report is in preparation including application of the ultimate strength method of design and the latest recommendations on volume change. (The previous two reports appeared in the March 1932 and September 1940 JOURNALS.)

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Massachusetts State Highway Dept.
CHARLES S. WHITNEY
Ammann and Whitney

*Nonmember ACI.
†Corporation Member.

Committee 314—Rigid Frames for Buildings and Bridges

Organized 1935

Mission—The assignment of this committee is to study reinforced concrete and prestressed concrete rigid frames with particular reference to various loadings and the effects of volume changes resulting from shrinkage, creep, and temperature change. Included in this scope of the work are theoretical and experimental research, and investigation of actual bridges and building frames.

Activities—The change in scope and the change in committee title from "Rigid Frame Bridges" to "Rigid Frames for Buildings and Bridges" was approved by the Board of Direction in Nov. 1957. Work during the past year has been on a 1/10-scale reinforced concrete model of an existing skewed rigid frame bridge and the development of a mathematical analysis for such structures. Previous work of Committee 314 has dealt mainly with tests on model beams and cylinders, on an aluminum model of a skewed-slab rigid frame bridge, and on a ⅛-scale model of a skewed slab geometrically similar to the ½-scale model tested previously at the University of Illinois. A report on this research, "Model Analysis of a Skewed Rigid Frame Bridge and Slab," by D. H. Pletta and Dan Frederick, was published in the ACI JOURNAL, Nov. 1954, *Proc.* V. 51, p. 217.

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Office, Chief of Engineers, Dept. of the Army

Committee 315—Detailing Reinforced Concrete Structures

Organized 1936

Mission—The committee's assignment is to prepare a report on detailing of reinforced concrete structures.

Activities—*Manual of Standard Practice for Detailing Reinforced Concrete Structures* (*ACI 315-57*) was published in 1957.

The manual presents recommended methods and standards for preparing drawings for the fabrication and placing of reinforcing steel in reinforced concrete structures. The previous ACI Standards ACI 315-51 (detailing of building structures) and ACI 315A-53 (detailing of highway structures) have been combined into one manual. Typical engineering drawings (and, for buildings, placing drawings as well) illustrate the use of the standards. There is also a chapter of instructions to designers.

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General Services Administration

Committee 317—Reinforced Concrete Design Handbook

Organized 1939

Mission—Committee 317 was reactivated in 1953 for the purpose of bringing up to date the ACI special publication *Reinforced Concrete Design Handbook* published in 1939.

Activities—The committee revised the handbook to conform with latest codes and practice; the second edition was made available late in 1955.

THOR GERMUNDSSON, Chairman
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Rail Steel Bar Assn.
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Phillips-Carter-Osborn, Inc.

Committee 318—Standard Building Code

Organized 1929

Mission—The committee's assignment is to review periodically and up-date the Institute's "Building Code Requirements for Reinforced Concrete."

Activities—The latest edition, ACI 318-56, was adopted at the 1956 annual convention and ratified by letter ballot. It was published in the May 1956 ACI JOURNAL (*Proc.* V. 52).

The 1957 work of the committee was devoted to two items: (1) The preparation of a question-and-answer paper explaining application of the appendix on ultimate strength design in ACI 318-56 (ACI JOURNAL, Sept. 1957, *Proc.* V. 54, p. 197). "Review of Changes in the ACI Building Code Requirements for Reinforced Concrete," by Frank Kerekes, formerly committee chairman, preceded the committee report in the September 1957 JOURNAL. (2) Developing a program for the next revision, including the creation of 15 subcommittees.

The committee will continue its standing assignment to improve the Code and provide safe minimum requirements for application of new methods of analysis, design, or construction and use of new materials.

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Office, Chief of Engineers, Dept. of the Army

*Nonmember ACI.

Committee 319—Recommended Practice for Use of Metal Supports for Reinforcement

Organized 1930
Discharged 1942

The report of this committee, "Recommended Practice for Use of Metal Supports for Reinforcement (ACI 319-42)," (June 1945 ACI JOURNAL) was dropped as an ACI standard Oct. 30, 1955. The current material contained in the report is now covered by ACI 315-57.

Committee 320—Safe Loading for Existing Concrete Bridges

Organized 1940
Discharged 1948

Committee issued a report, "Safe Loads for Existing Concrete Bridge," by F. W. Panhorst, author-chairman, ACI JOURNAL, January 1943.

Committee 321—Design of Reinforced Concrete Slabs—Joint ACI-ASCE

Organized 1941

Mission—This committee, joined as an ACI-ASCE endeavor in February, 1953, is assigned the problem of analysis and design of bridge and building slabs.

Activities—The immediate objective is the problem of floor slabs in buildings, including the effect of openings and concentrated loadings. The committee is making a study of accurate solutions of the problem that have been developed in recent years, as well as of experimental data that have been made available; this with a view toward investigating the adequacy of current and proposed approximate methods of design.

Recent activity has been the preparation of a research program on the relative strength and behavior of different types of floor slab construction.

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American Iron and Steel Institute

Committee 322—Design of Structural Plain Concrete

Organized 1942

Mission—Committee will review the 1942 report of the original committee, revise it, and expand it as necessary to provide a complete recommendation for the design of common unreinforced concrete applications. The ultimate objective is to develop a standard to supplement the "Building Code Requirements for Reinforced Concrete."

Activities—Reactivated in 1957, the proposed committee recommendations will include all subjects covered in the original report, "Proposed Recommended Stresses for Unreinforced Concrete," ACI JOURNAL, Nov. 1942, *Proc.* V. 39. It may be expanded to include as new sections, design requirements for plain concrete walls, plain concrete underground structures, basements, and thin precast elements, and to provide more completely allowable loads for unreinforced piers, and temperature and shrinkage reinforcement required for thick slabs, thick walls, and slabs on ground.

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University of Durham, England

*Nonmember ACI.
†Corporation Member.

Committee 323—Prestressed Reinforced Concrete—Joint ACI-ASCE

Organized 1942

Mission—This committee was assigned to review present knowledge of prestressed concrete, to develop design procedure, and to recommend needed research.

Activities—Reorganized as a joint ACI-ASCE group in 1952. "Tentative Recommendations for Prestressed Concrete" was published in the January 1958 JOURNAL. It constitutes a recommended practice, not a building code or specification. It is confined in scope to structural members involving linear prestressing. Most of the recommendations relate to flexural members and are intended to apply to both buildings and bridges.

A subcommittee has collected a bibliography consisting of over 2100 references, published from 1896 into 1955. A separate section lists patents. The second edition of *Bibliography on Prestressed Concrete* was published by ACI in 1955.

A subcommittee reviewed the patent situation on prestressed concrete, and prepared a paper, "Patents and Codes Relating to Prestressed Concrete," ACI JOURNAL, May 1950.

A subcommittee prepared a set of definitions and notations, "Proposed Definitions and Notations for Prestressed Concrete," ACI JOURNAL, October 1952.

The committee also sponsored a session at the 52nd annual convention in 1956 and the 53rd annual convention in 1957.

THOR GERMUNDSSON, Chairman
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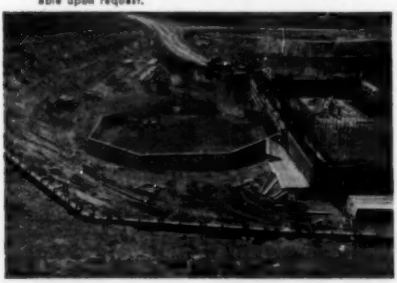

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Committee 324—Precast Reinforced Concrete, Thin Sections

Organized 1946

Mission—The mission of this committee is to prepare recommendations for design and construction utilizing thin section precast concrete.

Activities—The committee completed its preliminary assignment of a comprehensive survey of available information dealing with the design and construction techniques of thin section precast concrete structures. Utilizing the compiled data of the survey, the committee has submitted a report, "Tentative Recommendations for Thin-Section Reinforced Precast Concrete Construction," which was published in the May 1958 JOURNAL.

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*Nonmember ACI.
†Corporation Member.

Committee 325—Structural Design of Concrete Pavements for Highways and Airports

Organized 1948

Mission—The committee's assignment is to develop a recommended practice for the structural design of concrete highway and airport pavement—slab dimensions, cross section, joints, reinforcement, etc.—on the basis of the best information available, and to keep such recommendations up to date as new information indicates need for revision. These design recommendations complement the specifications for materials and construction prepared by Committee 617.

Activities—The ACI Standard "Recommended Practice for Design of Concrete Pavements (ACI 325-58)" was adopted at the 54th annual convention, ratified by letter ballot, and will be published in the July JOURNAL.

Reports of the four subcommittees were published in 1956: Subcommittee I—"Considerations for Construction of Subgrades and Subbases for Rigid Pavement," August 1956 JOURNAL; Subcommittee II—"Considerations in the Selection of Slab Dimensions," November 1956 JOURNAL; Subcommittee III—"Structural Design Considerations for Pavement Joints," July 1956 JOURNAL; and Subcommittee IV—"Design Considerations for Concrete Pavement Reinforcement for Crack Control," October 1956 JOURNAL.

The committee sponsored a session at the 54th annual convention in 1958 featuring the reports of three subcommittees on prestressed concrete pavements; use of continuous reinforcement in highway pavements; and design of concrete overlays for concrete pavements.

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Highway Research Board

Committee 326—Shear and Diagonal Tension —Joint ACI-ASCE

Organized 1950

Mission—A joint committee of ACI and ASCE was formed in 1950 with the assignment of developing methods for designing reinforced concrete members to resist shear and diagonal tension consistent with the new ultimate strength design methods.

Activities—An initial test program investigated shearing strength of a reinforced concrete slab under a centrally located concentrated load. A program of tests on the shear strength of concrete members without shear reinforcement, sponsored by the Reinforced Concrete Research Council, was recently completed at the University of Illinois.

The Committee on Reinforced Concrete Research of the American Iron and Steel Institute is also sponsoring an investigation at Columbia University on the diagonal tension resistance of reinforced concrete beams which are subjected to multiple loads simulating a uniformly distributed load.

Plans by the committee include an investigation of knee-frames to be performed of the University of Illinois also under the sponsorship of the Committee on Reinforced Concrete Research.

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AASHO Road Test

Committee 327—Ultimate Load Design —Joint ACI-ASCE

Organized 1952
Discharged 1957

A joint committee was formed in 1952 with the Subcommittee on Ultimate Load Design of the ASCE Committee on Masonry and Reinforced Concrete, Structural Division. The assignment was to evaluate and correlate theories and data bearing on ultimate load design procedures with a view to establishing them as accepted practice.

A report was issued in 1955 and published by both sponsoring organizations: "Ultimate Strength Design," Paper 809, ASCE *Proceedings*, V. 81, Oct. 1955; ACI JOURNAL, January 1956.

Portions of this report were abstracted to form the appendix to the 1956 ACI Building Code, which recognizes ultimate strength for the design of reinforced concrete members.

Committee 328—Limit Design— Joint ACI-ASCE

Organized 1956

Mission—The mission of this committee is to develop a recommended practice for the application of limit analysis to reinforced concrete structures. Its work will be an extension of that of ACI-ASCE Committee 327, Ultimate Load Design.

Activities—A comprehensive review has been prepared within the committee which will form a nucleus for subsequent theoretical and experimental studies by the joint committee. Theoretical studies on the amount of redistribution occurring with various percentages of reinforcement have been started.

These theoretical studies will be used as a guide for determining the amount of experimentation required.

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Committee 331—Structures of Concrete Masonry Units

Organized 1953

Mission—Committee 331 has been assigned the mission to develop methods of structural analysis and design, recommend design and construction practices, to provide users of concrete block, brick, and tile with authoritative information and guidance.

Activities—Subcommittee assignments are (1) structural design and specifications for materials and construction; (2) control of volume change cracking; and (3) design to utilize thermal, acoustic, and fire resistant properties.

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Committee 332—Recommended Practice for Residential Concrete Work

Organized 1953

Mission—Committee 332's initial mission was to prepare a recommended practice in the field of residential concrete work.

Activities—The committee is already actively engaged in studying the early publication of guide details on recommended good practice on residential work, with the first project, concrete floor slabs on grade, nearly completed. Subsequent committee work will take up types of simplified concrete foundations.

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Committee 333—Design and Construction of Composite Structures—Joint ACI-ASCE

Organized 1956

Mission—This committee is to study the integration of structural members of dissimilar materials in bridges and buildings. The objective is to prepare recommendations for the design and construction of structures composed of prefabricated beams combined with cast-in-place slabs.

Activities—Composite combinations using concrete include two subjects already under study. One is a research proposal on the connection between precast concrete beams and a cast-in-place slab; the other is a literature survey or bibliography on the bond between a steel beam and a concrete slab.

The committee intends to embark on a systematic study of composite construction. It is expected that such a study will consist of the review of several specific problems, e.g., working load design, ultimate load design, volume changes, shear connections, with the aim of collecting and correlating existing information and pointing out areas where research is needed. The next step would be to outline and execute research projects; and finally, the possible preparation of a manual for composite design and construction.

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Committee 334—Concrete Shell Structures

Organized 1957

Mission—The committee is to review present knowledge in the entire field of shell structures, including both theory and applications and to prepare an initial report on present practice in analysis, design, and specifications for materials and construction. Ultimately, the committee is to report recommended practice for design and specifications to supplement the provisions in the ACI Building Code as modifications necessary to suit the special conditions of shell structures.

The committee will consider all types of concrete shell structures and all materials and methods of construction of which applications are available for study. Design recommendations for cylindrical shell and folded plate roofs may be made by reference to reports of other organizations investigating these types. Recommendations for shell-type storage structures may be made by reference to reports of ACI Committee 714.

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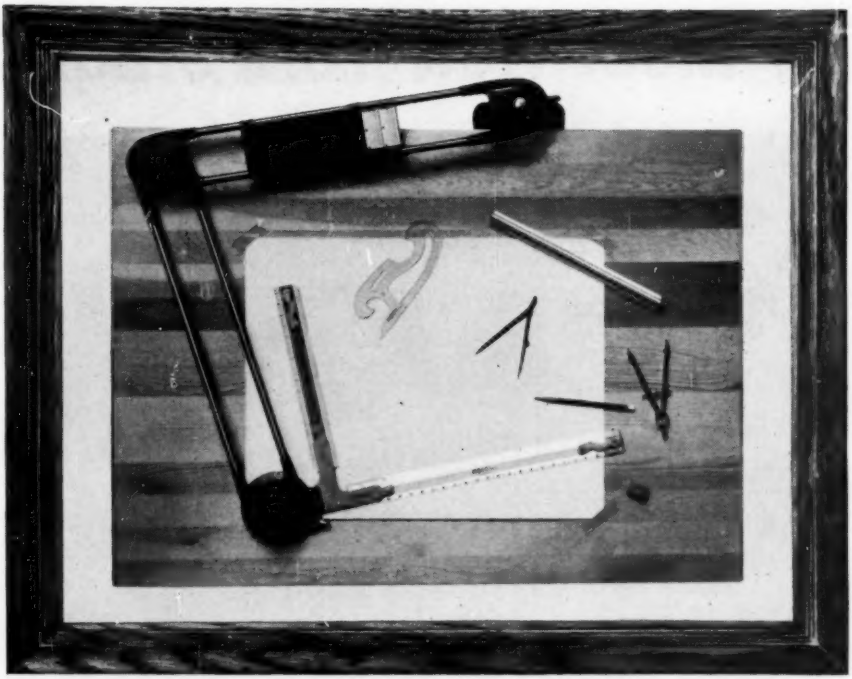
Committee 335—Deflection of Concrete Building Structures

Organized 1957

Mission—Committee 335 will study available research on deflection of concrete flexural members in building structures under rapid and long-term loads to develop recommendations for the prediction of such deflection. Suggested limits for allowable deflection of various types of concrete flexural members



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big office building

Cost: $28 million

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Short-span floors, reinforced with USS* American Welded Wire Fabric, are a feature of 100 Church Street, a brand-new twenty-story building now going up in Manhattan. Closely spaced beams support concrete floor slabs of minimum thickness. Short-span floors are strong and light in weight. And they save money because installation requires less time, labor, and material.

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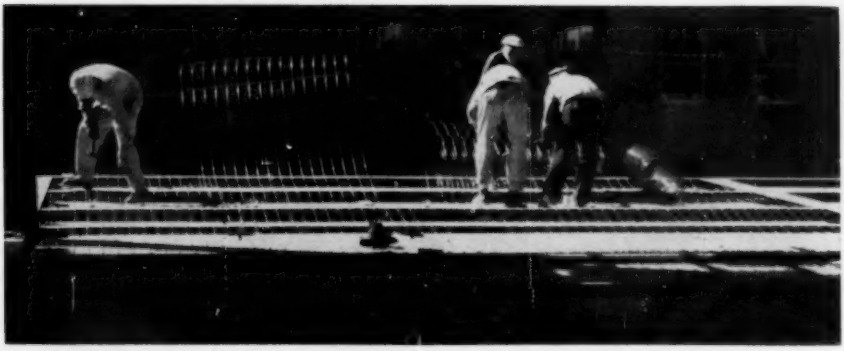

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Remember, buyers will ask, *"is it Reinforced"*

under various classes of loadings will be developed. The recommendations for prediction and limits of deflection shall be correlated with limitations on acceptable deflections, minimum depth requirements, and allowable limits on deflection and recovery under test loads prescribed in existing ACI Standards. First report is to cover existing satisfactory practice for ordinary structures and eventually to include all structures.

Activities—Field and laboratory research results will be studied. The studies will include effects of aggregate (lightweight or normal); high strength concretes; precasting-plant curing procedures; age and drying; continuity; amount, type, and location of reinforcement; prestressing; and composite action of precast with cast-in-place construction. Types of members will include beams, two-way and flat slabs, arches, folded plate and thin shell structures, and rigid frames. Classes of loading will include uniform and concentrated; wind, blast, or earthquake; moving; temporary; and permanent. Reference to and correlation with the reports of other committees will be expected.

This proposed scope is limited to building structures excluding bridges, mass concrete structures, tunnels, and storage structures. It could be further restricted to common building elements if desired, excluding arches, folded plates, and thin shells.

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A. C. Scordelis
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Mete Sozen
University of Illinois
Harry A. Williams
Stanford University

Committee 401—Specifications for Structural Concrete

Organized 1956

Mission—The committee is to prepare material to serve as a basis for the section on structural concrete in specifications. The report is to be in such form that applicable portions may be incorporated and additional special provisions inserted as required by the specification writer to form a complete specification for structural concrete. When generally accepted requirements in legal phraseology are available, i.e., appropriate sections of other ACI Standards or ASTM specifications, the provisions will be merely references. Other provisions may incorporate excerpts from generally accepted ACI reports and papers, including the Joint Committee Report of 1940, or recommendations based upon such generally accepted material may be prepared in legal phrasing by the committee. Suggested title for report is "Guide to Specifications for Structural Concrete."

George H. Nelson, Chairman
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Delmar L. Bloem
National Sand and Gravel Assn. and
National Ready Mixed Concrete Assn.
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Committee 402—Concrete Floor Finishes

Organized 1957

Mission—The committee will review all earlier reports of ACI committees and papers by individuals on concrete floor finishes, suggest a standard abrasion test, and prepare a recommended practice on materials and construction methods. The initial report of the committee will suggest a standard abrasion test and recommend research needed to establish a scale of performance ratings suitable for specifications. The final report of the committee will include a specification on concrete floor finishes supplementing the work of Committee 401—Specifications for Structural Concrete.

*Nonmember ACI.

The entire range of concrete floor finishes including heavy-duty types for exposure to industrial traffic wear and aggressive chemicals and exposed decorative floors will be studied. General classes of proprietary materials including metallic, carborundum, and other special aggregates and liquid chemical hardeners will be considered. A scale of performance ratings based on a standard wear test will be established and recommended ranges of ratings for various classes of use established.

GERALD MILSOM, Chairman
Metallicrete Floor Co., Ltd., Montreal

Committee 505—Design and Construction of Reinforced Concrete Chimneys

Organized 1922
Discharged 1956

The ACI standard, "Specification for the Design and Construction of Reinforced Concrete Chimneys (ACI 505-54)," was adopted at the 1954 annual convention, ratified by letter ballot in June, 1954, and published in the ACI JOURNAL, September 1954.

Committee 604—Winter Concreting Methods

Organized 1929
Discharged 1957

A revised standard was adopted at the 1956 convention and subsequently ratified by secret letter ballot. The new standard, "Recommended Practice for Winter Concreting (ACI 604-56)," was published in the June 1956 ACI JOURNAL; it superseded ACI 604-48.

Committee 605—Hot Weather Concreting

Organized 1955

Mission—This committee is to review information on the effect of hot weather on the characteristics of fresh and hardened concrete, and to develop a recommended practice for controlling ingredients, batching, mixing, transporting, placing, protecting, and curing concrete during hot weather.

Activities—A progress report, "Hot Weather Concreting Problems," was presented at the 1957 annual convention and published in the May 1957 JOURNAL (*Proc.* V. 53). The latest report of the committee has been submitted as a proposed recommended practice and is now in the process of consideration by the Standards Committee.

STANTON WALKER, Chairman
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National Ready Mixed Concrete Assn.
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BYRON P. WEINTZ
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JOHN J. WHITE
Parsons-Brinckerhoff-Hall and MacDonald
C. E. WUERPEL
Marquette Cement Mfg. Co.

Committee 609—Compaction of Concrete by Mechanical Means

Organized 1934

Mission—This committee has the assignment to revise and bring up to date a former report (ACI JOURNAL, Mar.-Apr. 1936, *Proc.* V. 32), and has been organized to encourage research to discover the basic effects within concrete produced by vibrators of different design, and to determine how their characteristics can be used to the greatest advantage.

Activities—The first draft of a recommended practice for consolidation of concrete by mechanical means is now being studied by the committee. A symposium on vibration practice was presented by the committee at the 1953 convention.

JOSEPH J. WADDELL, Chairman
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Jackson Vibrators, Inc.
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Waterways Experiment Station
ARTHUR A. LEVISON
Blaw-Knox Co.

*Nonmember ACI.

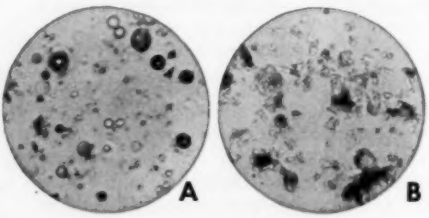
How much CEMENT...
How much FLY ASH...
How much AGGREGATE
...go into the best mix for a given requirement ?

THAT correct design of the mix is vital is not open to question. The best results are obtained when ingredients are proportioned in accordance with information obtained from extensive investigation. Programs of this kind are continuing. The information developed is available through the companies named below.

Fly Ash Improves Quality in Several Ways

A great many tests have shown that a concrete containing proper amounts of an approved fly ash is superior to a concrete made without fly ash . . . Fly ash concrete is easier to place and finish, and has a lower heat of hydration. It also has greater resistance to the penetration of water and to the effects of aggressive waters.

All these are important, depending on the purpose of the concrete. One quality *always* desired is high workability, ease of placing in the forms, filling them completely, using even less water, with a consequently denser mix and more watertight job. Main reason for the high workability is seen in microscopic views A and B. The particles of cement (B) are rough, angular, while those of fly ash (A) are mostly spherical and smooth. Thus the "ball bearing" effect which makes the mix handle more easily.



Each of the Companies Below Has Technical Data and Competent Engineers to Help You in Designing the Most Effective Mixes Employing Cement and Fly Ash

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WALTER N. HANDY COMPANY, INC.
P. O. Box 549, Evanston, Ill.

McNEIL BROTHERS, INC.
P.O. Box 4015, Bridgeport 7, Conn.

DETROIT EDISON COMPANY
2000 Second Ave., Detroit 26, Mich.

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Maginnis Power Tool Co.
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University of California
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Missouri River Div., Corps of Engineers
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California State Dept. of Water Resources
GEORGE W. WASHA
University of Wisconsin
HENRY A. WEIGAND
Southern Cen-Vi-Ro Pipe Corp.
GEORGE WERNER
Bureau of Public Roads

Committee 611—Inspection of Concrete

Organized 1935

Mission—The standing mission of this committee is to develop the *ACI Manual of Concrete Inspection* and keep it current by periodic revision. The mission also includes the promotion of effective inspection in every possible way through such means as encouraging greater recognition of inspectors and inspection, publication of reports on effective inspection, and encouragement of short courses and conferences on inspection.

Activities—The fourth edition of the manual was published in 1957. The committe has also fostered publication of papers on aspects of inspection in the ACI JOURNAL, *Western Construction* and *AIA Bulletin*; other papers are in preparation.

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Joseph K. Knoerle and Associates, Inc.
D. O. WOOLF
Bureau of Public Roads

Committee 612—Recommended Practice for Curing Concrete

Organized 1936

Mission—This committee is to prepare a recommended practice for curing concrete.

Activities—The committee is now studying a final draft of a report on curing. A proposed standard containing recommendations in specification form will be the next project. The committee has sponsored basic research at the National Bureau of Standards on curing methods, and a symposium on curing was sponsored at the 1952 convention.

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Huron Portland Cement Co.
MARK MORRIS
Iowa State Highway Comm.
F. V. REAGEL
Missouri State Highway Comm.
GEORGE WERNER
Bureau of Public Roads

Committee 613—Recommended Practice for Proportioning Concrete Mixes

Organized 1936

Mission—This committee has the assignment of developing recommended practices for proportioning concrete mixes, including air-entrained and non-air-entrained concrete, lightweight concrete, and no-slump concrete.

Activities—The committee has published the standard, "Recommended Practice for Selecting Proportions for Concrete (ACI 613-54)," (ACI JOURNAL, Sept. 1954), which superseded ACI 613-44.

A final draft of a recommended practice for proportioning structural lightweight concrete mixes has been prepared and early publication is anticipated. Another subcommittee is actively studying the problems involved in preparing recommendations for proportioning mixes for no-slump concrete, with particular reference to block, pipe, and other precast products.

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Waylite Co.

Committee 614—Recommended Practice in Measuring, Mixing, and Placing Concrete

Organized 1936

Mission—The committee is to review and bring up to date the standard "Recommended Practice for Measuring, Mixing, and Placing Concrete (ACI 614-42)," ACI JOURNAL, June 1945, and to consider inclusion of performance requirements for concrete batching and mixing plants.

Activities—The committee, discharged after completion of the 1942 standard, was reorganized in 1956. Rewriting of portions of ACI 614-42 has begun.

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Pittsburgh Testing Laboratories
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Winkworth Fuel and Supply Co.

*Nonmember ACI.
†Corporation Member.

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Bureau of Reclamation
D. K. WOODIN
East Bay Municipal Utility District

Committee 616—Coatings for Concrete

Organized 1936

Mission—The committee's assignment is to prepare recommended practices for the application of paint and coatings of different kinds to concrete surfaces, including industrial applications of coatings to prevent deterioration of concrete, dampproofing, and waterproofing.

Activities—The committee prepared the standard "Recommended Practice for the Application of Portland Cement Paint to Concrete Surfaces (ACI 616-49)," September 1949 JOURNAL. The report "Guide for Painting Concrete," for paints other than portland cement paint, was published in the March 1957 JOURNAL (*Proc.* V. 53).

A start has been made on preparation of a bibliography on the painting of concrete. Future assignments are: (1) industrial applications of coatings to prevent chemical attack on concrete, and (2) dampproofing and waterproofing of concrete.

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National Paint, Varnish, and Lacquer Assn.
FRANCIS SCOFIELD*
National Paint, Varnish and Lacquer Assn.
D. O. WOOLF
Bureau of Public Roads

Committee 617—Specifications and Recommended Practice for Concrete Pavements and Bases

Organized 1937

Mission—The standing assignment is to prepare and maintain current a specification for materials and construction of concrete pavements and also concrete bases used to support surface courses of other materials. The specification complements the recommendations for design by Committee 325.

Activities—The standard "Specifications for Concrete Pavements and Concrete Bases (ACI 617-58)," superseding ACI 617-51, was adopted at the 54th annual convention,

ratified by letter ballot, and will be published in the July 1958 JOURNAL.

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BAILEY TREMPER
California Division of Highways
STANTON WALKER
National Ready Mixed Concrete Assn. and National Sand and Gravel Assn.

Committee 621—Aggregates

Organized 1936

Mission—This committee was reorganized in 1955 with the mission of preparing a statement of information on aggregate that will evaluate properties of aggregate in terms of their influence on properties of concrete, discuss features of preparation and handling which have a bearing on concrete quality and uniformity, and in general, summarize what should concern the user about aggregate when he sets forth to do a first-class job.

Activities—Previous work of the committee yielded papers by committee members on production of sand and gravel, crushed stone, and slag.

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R. W. SPENCER
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*Nonmember ACI.
†Corporation Member.

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I. L. TYLER
Portland Cement Assn.
STANTON WALKER
National Sand and Gravel Assn. and National Ready Mixed Concrete Assn.
D. O. WOOLF
Bureau of Public Roads

Committee 622—Formwork for Concrete

Organized 1955

Mission—The committee's assignment is to recommend practice for concrete formwork in general terms, based on considerations of safety and appearance. Consideration is directed toward: (1) formulas for estimating maximum form pressure; (2) allowable stresses on form, shore, and scaffolding materials; accessories; soil; bracing and guys; etc., to be expressed as a fraction of the ultimate strength of the material; (3) allowable limits on deflection and/or strains in the above items; (4) allowable tolerances in general; (5) special limits for items (2), (3), (4) applicable to architectural concrete; (6) a suggested specification for use by architects and engineers; (7) special requirements for accessories to be used on architectural concrete; (8) forming at joints; (9) section dealing with special cases such as grouting in aggregate-filled forms, forming cast-in-place material composite with precast.

Activities—Subcommittee II's report, "Form Construction Practices," was published in the June 1957 JOURNAL (*Proc.* V. 53). Other subcommittees are engaged studying other aspects of forming.

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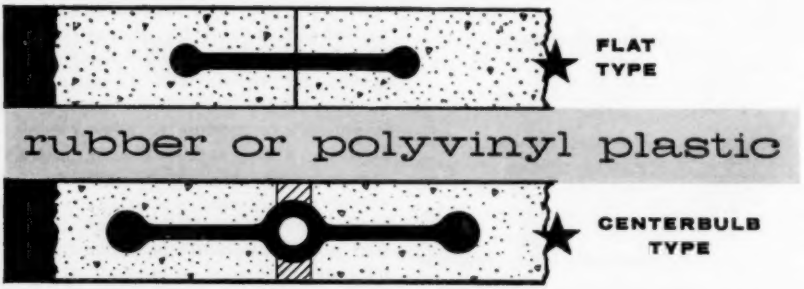

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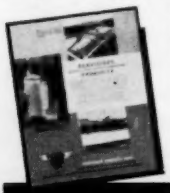

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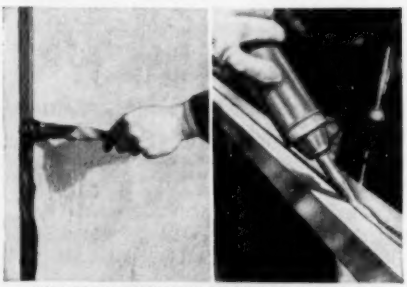

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WILLIAM R. WAUGH
Office, Chief of Engineers, Dept. of the Army
WILLIAM H. WOLF*
Bureau of Reclamation

Committee 623—Specifications and Practices for Foamed Concretes

Organized 1958

Mission—The committee will prepare recommendations for the use of foamed concretes. Specifications for materials and construction upon which such recommendations are based will be included. If necessary, special tests for compliance with the specifications may be suggested either for samples or for in-place construction.

A secondary standing assignment will be to furnish information on special properties of foamed concretes for use by Committee 213.

Activities—Now in the process of organizing, the committee will review research data including the report "Cellular Concrete," ACI JOURNAL, May and June 1954, *Proc.* V. 50, and later research results. Data on performance, construction practice, and design practice will be collected and evaluated. Tests for control and acceptance in use will be considered.

RUDOLPH C. VALORE, JR., Chairman
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Expanded Shale, Clay and Slate Institute
RICHARD J. FRAZIER
Anchor Concrete Products, Inc.
P. O. FREEMAN
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Mearl Mfg. Corp.

*Nonmember ACI.
†Corporation Member.

Committee 704—Cast Stone

Organized 1928
Discharged 1949

The report of this committee, "Specification for Cast Stone (ACI 704-44)," was published in the June 1945 ACI JOURNAL. The report was dropped as an Institute standard in 1956 because industry interest was insufficient to reactivate the committee to restudy the subject and keep the report current.

Committee 711—Precast Floor Systems for Houses

Organized 1934

Mission—The standing assignment is to prepare current recommendations for the use of factory produced precast concrete floor and roof units.

Activities—The latest report of this committee, "Minimum Standard Requirements for Precast Concrete Floor and Roof Units (ACI 711-58)," superseding ACI 711-53, was adopted at the 54th annual convention, ratified by letter ballot, and will be published in the July 1958 JOURNAL.

LEO M. LEGATSKI, Chairman
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Libbey-Owens Ford Glass Co.
R. N. VAKIL
University of Wisconsin

Committee 712—Precast Structural Concrete Design and Construction—Joint ACI-ASCE

Organized 1957

Mission—The mission of the committee is to develop a recommended practice to cover the over-all problems of design and assembly of beams, slabs, columns, and other precast

concrete members. Under consideration is the inclusion of precast prestressed members (particularly pretensioned), lift-slab, and tilt-up construction.

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Anderson, Birkeland and Anderson
MAURICE BENDER
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Consulting Engineer
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A. C. SCORDELIS
University of California

Committee 714—Concrete Bins and Silos

Organized 1937

Mission—This committee is to prepare recommendations for the design and construction of concrete structures for the storage of solids. The report is to include data on static design pressure for common solids, such as cement, flour, sugar, grain, and coal. Design recommendations will cover use of prestressed concrete and will include folded-plate analysis for circular and prismatic structures, allowance for dynamic pressure in loading and unloading, foundations to resist overturning from lateral or unbalanced loads, structural details to prevent local failure, multi-cellular and single units, and any modifications of the ACI Building Code necessary to suit special conditions in such storage structures. Recommendations for specifications will include any necessary provisions peculiar to such storage structures and any special protective treatments required for storage of particular common bulk materials. The recommendations will include any special precautions, limitations, and requirements for the sliding-form or shotcrete methods of construction.

Activities—With its reactivation in 1956, the committee's title and mission were changed to that above. The original committee, discharged in 1948, prepared the ACI Standard "Recommended Practice for the Construction of Concrete Farm Silos (ACI 714-46)," published in the October 1946 ACI JOURNAL.

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Chalmers and Borton
WILLIAM L. CLARK, JR.
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Jones-Hettelsater Construction Co.
PAUL ROGERS
Paul Rogers and Associates
KEEVE STEYN
Structural Engineer
LOUIS E. VANDERGRIFT
Ohio State University
ROBERT J. VAN EPPS*
Portland Cement Assn.

Committee 716—High Pressure Steam Curing

Organized 1941

Mission—The committee was assigned the task of studying, correlating information, and reporting on the subject of high-pressure steam curing.

Activities—Reactivated in 1950 because of an awakening of interest in high-pressure steam curing, the new group began a study of the report of former Committee 716 (ACI JOURNAL, April 1944, *Proc.* V. 40, p. 409) considering possible additions to the old report.

Committee work has resulted in reports published in the April and May, 1953, issues of the JOURNAL ("Physical Properties of High-Pressure Steam-Cured Concrete Block" and "Chemical Reactions in High-Pressure Steam Curing of Portland Cement Products," respectively).

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Alpha Portland Cement Co.

*Nonmember ACI.
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Owens-Illinois Technical Center
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True Gun-All Equipment Corp.
L. G. VINSON
Vinson Construction Co.
STANLEY G. ZYNDA
Gunite Contractors Assn.

Committee 805—Application of Mortar by Pneumatic Pressure

Organized 1942

Mission—Committee will review the existing standard "Recommended Practice for the Application of Mortar by Pneumatic Pressure (ACI 805-51)" [ACI JOURNAL, May 1951, *Proc.* V. 47] and prepare a revision. Objectives of the revision will be to reflect new developments in this field since 1951 extending the scope of the original report as necessary and to expand sections presently requiring clarification.

Activities—The committee, reactivated in 1957, will extend the scope of the report to include limits of applicability and recommendations for use of plastic mix shotcrete, for admixtures suitable in shotcrete, for lightweight aggregate shotcrete, for use of cements other than portland, and for construction of thick structural members.

J. J. CLOSNER, Chairman
Preload Co., Inc.

Joint Committee

Organized 1904
Discharged 1942

The first Joint Committee was formed in 1904 at the beginning of reinforced concrete construction in the United States. A second was formed in 1920 and a third in 1930. The report of the last Joint Committee, "Recommended Practice and Standard Specifications for Concrete and Reinforced Concrete," was published in 1940. Represented on the committee were ACI, American Institute of Architects, American Railway Engineering Assn., American Society of Civil Engineers, American Society for Testing Materials, and Portland Cement Assn.

A joint ACI-ASCE steering committee was formed in 1953 to study the 1940 report with a view to future action on it. The committee recommendation that the 1940 Joint Committee Report be discontinued was approved by both organizations in 1956, and all affiliated groups were notified.

ACI participation in the work of committees of other organizations

ASA Sectional Committee (A1) Specifications and Methods of Test for Hydraulic Cements

MILES N. CLAIR
Thompson and Lichtner Co., Inc.

ASA Sectional Committee (A37) Road and Paving Materials

HAROLD ALLEN
Bureau of Public Roads

*Nonmember ACI.

ASA Sectional Committee (A41) Masonry

WALTER J. McCOY
Lehigh Portland Cement Co.

ASA Sectional Committee (A42) Plastering

WALTER J. McCOY
Lehigh Portland Cement Co.

ASA Sectional Committee (Z23) Sieves

W. R. WAUGH
Office, Chief of Engineers, Dept. of the Army

ASTM Committee (C1) Cement

THOMAS B. KENNEDY
Waterways Experiment Station

ASTM Committee (C-9) Concrete and Concrete Aggregates

HARRY F. THOMSON
Engineering Consultant

Subcommittee I-d, Editorial and Definitions, ASTM Committee (C9) Concrete and Concrete Aggregates

EDWARD E. BAUER
University of Illinois

ASTM Committee (E5) Fire Tests of Materials and Construction

G. E. TROXELL
University of California

American Welding Society (Subcommittee on Welding of Bars, Metal Inserts and Connections in R/C Construction)

PHIL M. FERGUSON
University of Texas

Concrete Industry Board (New York)

ROGER H. CORBETTA
Corbetta Construction Co.

Concrete Reinforcing Steel Institute Committee on Awards

VERNON P. JENSEN
C. F. Braun & Co.

Construction Specifications Institute

PAUL F. RICE
American Concrete Institute

Highway Research Board

E. A. FINNEY
Michigan State Highway Dept.

International Federation of Prestressed Concrete

CURZON DOBELL
Preload Enterprises, Inc.

Reinforced Concrete Research Council

CHARLES S. WHITNEY
Ammann and Whitney

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Positions and Projects

ACI Building Code translated into German

A copy of *Sammlung von Bauvorschriften für Stahlbeton*, a German translation of "Building Code Requirements for Reinforced Concrete (ACI 318-56)" has been received from the Deutscher Beton Verein, Wiesbaden. Marginal notes in the German version refer to corresponding provisions in German engineering standards so that comparisons may be made readily. The German Concrete Association prepares translations of concrete regulations from different countries for use by its members who may engage in construction abroad.

Swiss group visits ACI

A group of Swiss architects and engineers on a study trip in the United States visited the new ACI headquarters building during their recent 2-day stay in Detroit. The group saw as well the General Motors Technical Center, the Ford Motor Co. office building, Northland shopping center, and the Detroit Civic Center. They were also briefed on Detroit's expressway and parking programs.

Arthur J. Tennenbaum of Giffels and Rossetti, Detroit, served as their guide and interpreter.

University of Colorado structural conference

The structural engineering conference held April 25 at University of Colorado was devoted to reinforced concrete floor systems. Among the speakers on the program were: George Hansen, Sallada and Hansen; Ib Falk Jorgensen, consulting engineer; Samuel R. Judd, U. S. Bureau of Reclamation; Claude C. Klemme, Jr., Commercial Testing Laboratories; William C. Muchow, of Berne, Muchow, Baume and Polivnik; and Chet Schrepferman, Brown and Schrepferman, all of Denver.

Dutton Biggs, Portland Cement Association, Kansas City, Mo.; James Chinn, University of Colorado, Boulder; and C. L. Garrett, Portland Cement Association, Oklahoma City, also participated in the program sponsored by the department of civil engineering, University of Colorado, in cooperation with the Portland Cement Association.

Past president Moreell honored by engineers

The Washington Award was recently presented to ACI past-president Ben Moreell, chairman of the board of Jones and Laughlin Steel Corp., in recognition of "distinguished service as a skilled engineer, outstanding naval officer, industrialist, and Hoover Commission associate." The award is administered by the Western Society of Engineers on recommendation of a commission representing the American Society of Civil Engineers, the American Institute of Electrical Engineers, American Institute of Mining, Metallurgical and Petroleum Engineers, American Society of Mechanical Engineers, and Western Society of Engineers.

During his nearly 30 years of naval service, Admiral Moreell rose from the ranks to become the only non-Annapolis graduate ever to attain the permanent rank of admiral. When he retired from active service in 1946 he channeled his energies into directing Jones and Laughlin Steel Corp., where he launched a $500,000,000 expansion program.

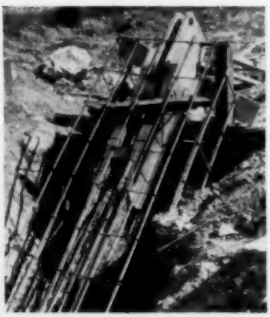
Admiral Moreell became a member of the Institute in 1930, a member of the board in February 1935 and president in 1941. He served 12 years on the Institute's board of direction.

Eastern European research data made available

Pergamon Institute, a nonprofit foundation, has recently been formed in Washington, D. C., and is in course of formation in London, for the purpose of making available to English-speaking scientists, doctors, and engineers (from all countries that are members of the United Nations), the results of scientific, technological and medical research and development in the Soviet Union and other countries in the Soviet orbit. Persons desiring information on what is being published in the U.S.S.R. and neighboring countries may ap-

TV tower foundations and anchorages strengthened with LACLEDE PREFABRICATED REINFORCING

KSD-TV's new 1152-foot television antenna tower in St. Louis, Missouri, is held upright by fifteen 2" guy cables secured to six massive underground concrete blocks. Each required approximately 100 cubic yards of concrete. Laclede reinforcing steels used in the anchors were shop weld-assembled to insure accurate fabrication, and proper distribution of the heavy design load requirements imposed upon the tower and its guys.



Delivered to the job site ready for installation, the reinforcing steel for each of the anchors was prefabricated and welded into units at Laclede's plant to insure maximum strength.

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CONSULTING ENGINEER: Wm. C. E. Becker
CONTRACTOR: E. C. Mikkelsen Construction Company, Inc. All of St. Louis, Missouri

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ply to the Institute to receive, free of charge, a monthly contents list in English of all the significant articles and books being currently published in his field of interest in these countries. Full English translation of any article mentioned in these contents lists may be ordered, and the charge for such a translation will be on a cooperative cost-sharing basis.

Pergamon offices are at 122 E. 55th Street, New York, N.Y.

AASHO Road Test construction resumed

Construction has been resumed at the AASHO Road Test at Ottawa, Ill. The $22-million highway research project will test widely varied thicknesses of pavement under truck axle loads ranging from 2000 to 30,000 lb on single axles and 24,000 to 48,000 lb on tandem axles.

The test facility, six loops containing 836 separate test sections, will be completed and ready for traffic in the late summer. The test sections will have nearly 200 different combinations of thicknesses of surfacing and underlying layers of material.

WANTED

ACI *Proceedings* V. 37

Sept. 1940 ACI JOURNAL

Feb. 1941 ACI JOURNAL

ACI Publications Department

Traffic will run in 10 lanes in five test loops about 18 hours a day, 6 days a week for 2 years. A complete system of instruments, many developed specifically for the project, will help measure and record the effects of the traffic on the pavements.

The project is sponsored by the American Association of State Highway Officials, and administered by the Highway Research Board of the National Academy of Sciences—National Research Council. Construction of the test facility is being directed by the Illinois Division of Highways with a special task force under the supervision of W. E. Chastain, Sr., engineer of physical research.

Argentina engineer inspects ACI headquarters

Ernesto Garcia Ollno, Buenos Aires engineer, visited ACI headquarters recently. Mr. Ollno reported that Argentine engineers and builders are greatly interested in the building and material standards of the United States.

Mr. Ollno, representing the Division Tecnica de Vivienda y Urbanismo del Centro Argentino de Ingenieros and Fundacion Migone del Centro Argentino de Ingenieros, has been touring the United States to study building standards and specifications, new materials, and construction techniques, particularly those that could be applied in the construction of low cost housing in Argentina.

Revised concrete mixer standard

The Associated General Contractors of America has released the 23rd revision of its concrete mixer standard. This standard was first prepared by the Mixer Manufacturers Bureau, an affiliate of the AGC, in 1924 and now includes specifications for construction mixers, paving mixers and plaster and mortar mixers.

The 1958 officers of the Mixer Manufacturers Bureau include A. K. Thomas, Chain Belt Co., chairman and W. A. Clayton, T. L. Smith Co., vice-chairman.

Vermiculite Institute 17th annual meeting

Confidence in the nation's economy and a bright future for the construction industry was the keynote of the 17th annual meeting of Vermiculite Institute recently in Chandler, Ariz. J. Brooks Robinson, vice-president of Robinson Insulation Co., Great Falls, Mont., was elected president. Two new directors were also named: Shelton W. Greer, president of Vermiculite Products, Inc., Houston, Tex., and L. G. McDiarmaid, president of Insulation Industries (Canada), Ltd., Vancouver, B. C.

C. R. Babb, chairman of the Vermiculite Institute's concrete and roofing committee, discussed a roof system that shows promise for schools and similar buildings. It consists of concrete joist framing with a steel welding

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strip cast into the top chord of the joist. Galvanized steel sheets are welded to the strip, and a 1:6 mix of vermiculite concrete is placed to desired thickness. The under side of the steel is sprayed with vermiculite acoustical plaster for noise control and a favorable fire rating.

Thus far the system has been used primarily in southern areas of the nation. One major advantage is that any type of conduit can be run on top of the metal sheets and covered with concrete, Mr. Babb said.

Folded plate seminar in Michigan

The Michigan section of the American Society of Civil Engineers in conjunction with the Portland Cement Association presented a seminar on folded plate structures in four consecutive weekly lectures May 1-22 in Detroit. Subjects covered included basic theory of folded plates, analysis of folded plates, construction methods, and model analysis.

Among those participating in the program, most of whom are ACI members were: Elihu Geer, University of Detroit; J. Gardner Martin, Portland Cement Association, Lansing; Linn Smith of Smith, Tarapata and MacMahon, Inc., Birmingham, Mich., and W. C. Krell, PCA, Detroit.

Others on the lecture program included Dudley Newton, Wayne State University, Detroit; A. L. Parme, PCA, Chicago; and L. K. Welch, Eberle M. Smith and Associates, Detroit.

The last of the lecture series featured J. R. Strang, Pulte-Strang, Inc., Ferndale, Mich.; W. L. Baxter and A. A. Shmina, both of Shmina and Sons, Dearborn, Mich.; J. M. Paulson, Wayne State University, Detroit; and L. C. Maugh, University of Michigan, Ann Arbor.

Conference of Canadian building officials

The annual conference of the Building Officials of Canada was held April 14-16 at Ottawa. Highlight of the 3-day meeting was a talk by Walter Bowker of the Federal District Commission on the National Capital Plan, followed by a tour of FDC projects in the national capital area.

The Division of Building Research, National Research Council (Canada), conducted a question-and-answer session on application and interpretation of sections of the National Building Code and presented a paper on new building materials and methods of construction, particularly in the fields of curtain wall construction, cellular concrete, and plastics.

Concrete leaders visit Germany and Russia

ACI past-president, Walter H. Price, head of the engineering laboratories, U. S. Bureau of Reclamation, Denver, recently returned from Europe where he attended the Third International Congress on Prestressed Concrete held May 5-10 in Berlin. Mr. Price also spent ten days touring Russia, visiting construction sites where large shell and large panel, precast and prestressed concrete were being used. Returning to Belgium on May 21, Mr. Price attended the International Exhibition at Brussels before his return to the United States.

Others who attended the prestressing congress and made the Russian tour included: Ben C. Gerwick, Jr., Ben C. Gerwick, Inc., San Francisco; James D. Piper, Portland Cement Association, Chicago; David P. Billington, Roberts and Schaefer Co., New York; T. Y. Lin and Boris Bresler, both of the University of California, Berkeley. Professor Lin acted as head of the American delegation.

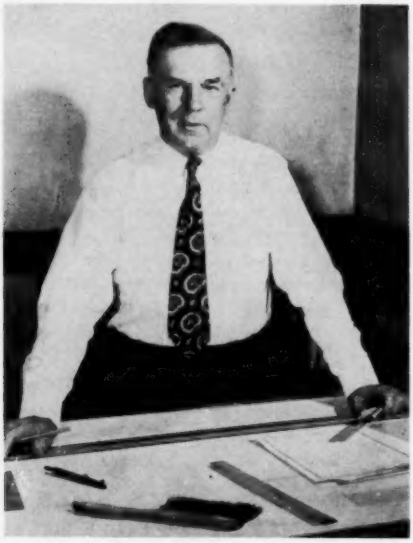
PCA opens Rocky Mountain regional office

Establishment of a new regional office in Denver, Colo., has been announced by G. Donald Kennedy, president of the Portland Cement Association. Edward W. Thorson, in charge of the association's district office in Denver since 1946, was appointed regional manager-district engineer effective Apr. 1, 1958. In this position he will supervise district offices in Salt Lake City, Utah, and Helena, Mont., in addition to Denver.

Mr. Thorson has been with PCA continuously since 1934 except for 4½ years service as a naval officer during World War II. He is a member of ACI and currently is president of the Colorado chapter of the National Society of Professional Engineers.

Newlin Dolbey Morgan

Newlin Dolbey Morgan, Sr., professor emeritus of architectural engineering, University of Illinois, died in Laramie, Wyo., March 7. A graduate of the University of



Colorado in 1910, he received his MS and CE degrees from the University of Illinois.

Teaching at the University of Illinois constituted the major portion of his professional life, where for 30 years he worked among students, from 1924 to 1928 in the civil engineering department and from 1928 until his retirement in 1954 in the department of architectural engineering. During this time he was a member of a number of professional and honorary societies including ACI, American Institute of Architects, American Society of Civil Engineers, Sigma Xi, and Tau Beta Pi.

Professor Morgan was one of ACI's most prolific member-getters. He is credited with securing over 250 new members for the Institute.

In June 1955 the College of Fellows of the American Institute of Architects issued a citation honoring him for meritorious teaching. A recognized authority in the field of concrete, two of his best known books are *Continuous Frames in Reinforced Concrete* and *Statically Indeterminate Structures*, which he co-authored with Professor Hardy Cross, Yale University.

Alden K. Fogg

Captain Alden K. Fogg, CEC, United States Navy, retired, died April 25 at San Diego, Calif. Born in Lowell, Ill. in 1892, Captain Fogg graduated from the University of Illinois in 1915. He was commissioned in the navy in 1917, and served in the Civil Engineer Corps with great distinction until his retirement in 1948. He was awarded the Legion of Merit and Letter of Commendation for his outstanding performance of duty during World War II.

After retirement Captain Fogg became city engineer for San Diego. He was a member of the American Concrete Institute, American Society of Civil Engineers, and the American Public Works Association.

Rudolph Saliger, European pioneer in reinforced concrete

Rudolph Saliger, one of the last surviving European pioneers of reinforced concrete died Jan. 31, 1958 in Vienna, Austria. He obtained his doctor's degree from the Technical University in Vienna in 1903 on the basis of his dissertation "Stress Conditions of Materials with Variable Elasticity, Especially of Reinforced Concrete." In 1908 he was appointed associate professor at the Technical University in Prague, introducing new classes on reinforced concrete and structural analysis. In 1916 two of his courses, reinforced concrete for civil engineers, and theory of reinforced concrete structures, started at the Technical University in Vienna.

Professor Saliger published 30 books as well as some 300 technical papers; his books have been translated into several languages. He had been widely honored for his achievements as engineer, teacher, and scientist. After his election as dean of the department of civil engineering he became president of the Technical University in Vienna in 1924. Other honors followed, including membership in the Academies of Sciences in Prague and Vienna.

Who's Who This Month

Frank H. Jackson

"Long-Time Study of Cement Performance in Concrete. Chapter 11—Report on the Condition of Three Test Pavements After 15 Years of Service" appearing on p. 1017 was prepared by Frank H. Jackson, engineering consultant, Chevy Chase, Md.

Past president of ACI, Mr. Jackson is well known to JOURNAL readers and has been active in Institute affairs since 1924. He was awarded, along with Harold Allen, the ACI Wason Medal for "the most meritorious paper" in 1948 for "Concrete Pavements on the German Autobahnen." Associated with the Bureau of Public Roads from 1905 until his retirement in 1953, he has received recognition from several organizations for his work in the highway field, including the Distinguished Service Award of the Highway Research Board in 1948 and the Meritorious Service Award of the U. S. Department of Commerce in 1950.

Mr. Jackson is an honorary member of ACI and of the American Society for Testing Materials. He currently is serving on ACI Committee 201, Durability of Concrete in Service; Committee 212, Admixtures; and Committee 613, Recommended Practice for Proportioning Concrete Mixes.

Tien S. Chang and Clyde E. Kesler

Co-authors Chang and Kesler have studied "Static and Fatigue Strength in Shear of Beams with Tensile Reinforcement" presented on p. 1033.

Tien S. Chang is a graduate of the University of Illinois where he spent the last 5 years on the experimental and theoretical studies of viscoelasticity, fatigue and creep of concrete, and rheology. He is presently an associate professor of the department of applied mechanics at the Virginia Polytechnic Institute, Blacksburg, where he is now actively engaged in the research of sonic testing, supersonic compressors, and magnetohydrodynamics. He is a member of ACI, ASTM, and other professional societies.

Clyde E. Kesler, professor of theoretical and applied mechanics, University of Illinois, Urbana, has had many technical papers published describing research in fatigue, creep, and sonic behavior of concrete. He graduated from the University of Illinois and has served as a consultant on industrial problems and research. He is a member of many honorary and professional groups including ACI, American Society of Civil Engineers, and ASTM.

Active in ACI committee work, Professor Kesler is now a member of Committee 209, Volume Changes and Plastic Flow in Concrete and Committee 215, Fatigue of Concrete. He also serves on the Institute's Technical Activities Committee.

Paul J. Fluss

"Uniform Structural Lightweight Aggregate Concrete Through Careful Proportioning and Control," on p. 1059, is the work of Paul J. Fluss, testing engineer, San Francisco Port Authority, San Francisco.

After studying chemical engineering at the Institute of Technology in Vienna, Mr. Fluss was engaged in the asphalt and highway construction industry in Austria from 1922 until the outbreak of World War II. He came to this country in 1940 where he worked in the varnish and rubber industry, doing research and testing.

In 1947 he became testing engineer of the San Francisco Port Authority. In this capacity he designs, tests, and evaluates all materials used for marine construction, concrete being the most important of the materials investigated. Mr. Fluss joined the Institute in 1953.

Paul Klieger

Another research paper prepared by Paul Klieger, "Effect of Mixing and Curing Temperature on Concrete Strength," appears on p. 1063.

Mr. Klieger is senior research engineer, applied research section, Portland Cement Association, Chicago. After graduating from the University of Wisconsin in 1939, he worked

for a short time with the Soil Conservation Service of the U. S. Department of Agriculture. In 1941 he joined the research staff of PCA. Army service interrupted his career from 1942 to 1945, but he then returned to PCA research and development laboratories.

Mr. Klieger, a member of ACI, has had a number of technical papers published in the ACI JOURNAL and the Highway Research Board's *Proceedings* and *Bulletin*. He is a member of ASTM Committee C-9 on Concrete.

Andrew Gallia

"Precast Reinforced Concrete Slab Bridges with Stiffened Edges," p. 1083, was submitted by Andrew Gallia, senior civil engineer, department of public works, Hobart, Tasmania, Australia.

After graduation in civil engineering from the University of Berlin, Germany, in 1924, Mr. Gallia engaged in varied engineering work in Europe, North Africa, and Persia. In 1953 Mr. Gallia had the opportunity of studying prestressed concrete bridges in France, Belgium, Switzerland, and England. He is a member of ACI, the Institution of Engineers, Australia, and the International Association for Bridge and Structural Engineering.

Mr. Gallia has been carrying out extensive research work on the distribution of wheel loads on highway bridges with particular reference to bridges with stiffened edge members, and several of his articles have been published in technical journals here and abroad.

C. E. Lovewell and George W. Washa

"Proportioning Concrete Mixtures Using Fly Ash," p. 1093 was co-authored by C. E. Lovewell and George W. Washa.

C. E. Lovewell, sales manager and service engineer, Chicago Fly Ash Co., Chicago, is a graduate of the University of Wisconsin and has participated in various research programs for many years. In the thirties he was engaged in research at the National Bureau of Standards; later he was field engineer for the New York and New Jersey Brick Manufacturer's Association. He then spent 9 years with the Louisville Cement Co., further sharpening his interest in concrete, concrete products, and research. Mr. Lovewell then became director of research and chief engineer for the Carney Co., Mankato, Minn., leaving this organization in 1951 to make his present affiliation.

He is now engaged in two research programs: one, in association with Professor Washa at the University of Wisconsin, and the other in association with Prof. P. W. Manson, University of Minnesota. Mr. Lovewell joined ACI in 1956.

G. W. Washa is professor and chairman of the department of mechanics at the University of Wisconsin, where he graduated in 1938. An active ACI member since 1940, Professor Washa has been a frequent contributor to the JOURNAL. In 1940 he was awarded the Wason Medal for Research for work reported in his paper "Comparison of the Physical and Mechanical Properties of Hand Rodded and Vibrated Concrete Made with Different Cements."

Professor Washa, a former director of the Institute, is chairman of ACI Committee 115, Research, and a member and former chairman of Committee 213, Properties of Lightweight Aggregates and Lightweight Aggregate Concrete. He is also a member of ACI Committee 609, Compaction of Concrete by Mechanical Means, and Committee 613, Recommended Practice for Proportioning Concrete Mixes.

Earl B. Cohn and W. A. Wall

"Military Personnel Records Center Built Without Expansion Joints" is described on p. 1103 by Earl B. Cohn and W. A. Wall.

Earl B. Cohn, head of structural design unit, Kansas City District Office, Corps of Engineers, is in charge of supervising structural design for dams and buildings prepared in the district office; he also reviews structural design prepared by architect-engineers for buildings used at military installations.

Mr. Cohn's experience includes work with American Bridge Co. and Missouri Pacific Railroad Co. before joining the Corps of Engineers in 1934. In his present position he has supervised design of Harlan County Dam, Tuttle Creek Dam, and Pomme de Terre Dam. In addition to the Military Personnel Records Center, he has reviewed

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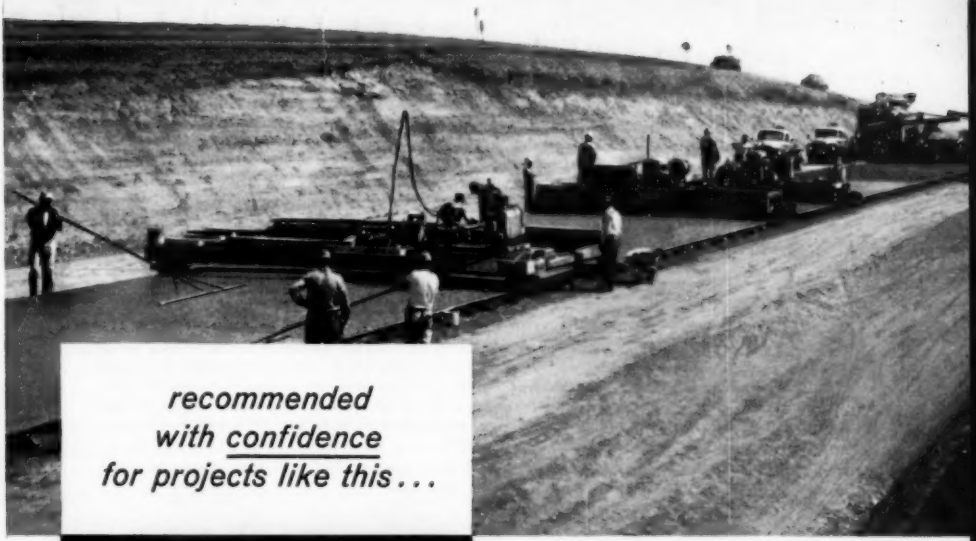

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Above: Section of U. S. Route 40, west of Topeka. Kansas State Highway Commission. Contractor: Koss Construction Co.

Below: Route 5, approximately 5 miles east of Erie. Pennsylvania State Highway Department. Contractor: Geo. S. Mellert-Weidner Company.

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design of buildings such as the hospital at Fort Riley, Kan., and the Academic Building, Fort Leavenworth, Kan.

W. A. Wall is head of the architectural unit of the Kansas City District Office, Corps of Engineers. He graduated from Montana State College and started his career with the Corps of Engineers in 1934 as a civilian employee on the world's largest hydraulic earth fill dam at Fort Peck, Mont. Later he was office engineer for the Kanopolis Dam at Ellsworth, Kan. Since 1942 he has been in the Kansas City office acting in various capacities of design, research, and administration of civil and military projects. Besides the Military Personnel Records Center, he has been design project engineer for the A. G. Publication Center, St. Louis, Mo.; the Kansas City Veterans Administration Hospital; Command and General Staff College, Fort Leavenworth; all types of military construction, including enlisted men's housing; office quarters, and other station facilities at Fort Riley and Fort Leavenworth, and at Fort Crowder and Fort Leonard Wood in Missouri.

A. M. Freudenthal and Frederic Roll

Combining their research efforts, A. M. Freudenthal and Frederic Roll have coauthored a paper, "Creep and Creep-Recovery of Concrete Under High Compressive Stress" appearing on p. 1111.

A. M. Freudenthal, professor of civil engineering at Columbia University, New York, has been on the staff there since 1949. Born in Poland, Dr. Freudenthal graduated from the Technische Hochschule in Prague, Czechoslovakia, and received the Doctor of Engineering Sciences degree in 1930. In the same year he completed the curriculum in theoretical physics at Charles University, Prague. He has worked as design engineer and superintendent of construction on various reinforced concrete projects in Czechoslovakia, Poland and Palestine. During the war he was consultant to the chief engineer of the British Forces in Palestine and Transjordan. Prior to coming to the United States in 1947, Dr. Freudenthal was associated for 10 years with the Hebrew Institute of Technology, first as lecturer, later as professor of civil engineering.

He came to the United States in 1947, joined the faculty of the University of Illinois, then practiced as a civil engineer in New York, before assuming his present duties.

Author of a number of technical papers and handbooks which have been published in this country and Europe and of a monograph on composite columns in reinforced concrete, Dr. Freudenthal was awarded the Norman Medal of the American Society of Civil Engineers in 1949 and again in 1958. His textbook, *Inelastic Behavior of Materials and Structures*, published in 1950, has been translated into German and Japanese.

Frederic Roll is associate professor in civil engineering at the University of Pennsylvania, Philadelphia. A graduate of Columbia University, he previously taught at the College of the City of New York and Columbia

LOS ANGELES IN '59

ACI's Technical Activities Committee is already looking forward to the 1959 convention in Los Angeles; program plans are taking shape.

Those who intend to offer papers for presentation at the 55th annual meeting should write to Institute headquarters before July 1, furnishing a synopsis which should make clear the scope of the proposed paper and indicate features that the author thinks will justify its inclusion in the program. Contributors should have preliminary drafts of manuscripts in the hands of the Technical Activities Committee for appraisal and acceptance by Sept. 15, 1958, and final manuscripts of accepted papers will be due Jan. 1, 1959.

From the replies received and suggestions from other sources, TAC will select papers to make up each convention session.

Write Institute headquarters NOW about the papers YOU want to see on the 1959 program.

University. He has also worked in private engineering firms as a stress analyst and highway and highway bridge designer.

Dr. Roll has conducted numerous research projects and industrial tests many of which have been specific studies in reinforced and plain concrete. He is a member of ACI and several other professional societies and is currently a member of ACI-ASCE Committee 326, Shear and Diagonal Tension.

Roebling appoints Stansbury

Lloyd F. Stansbury has been made West Coast supervisor for the construction materials division, John A. Roebling's Sons Corp., replacing the late Robert J. Cole.

Mr. Stansbury will be in charge of the division's sales in the 11 western states, Alaska, and the Hawaiian Islands. He has been associated with the Seattle district office of the Roebling firm for the past 8 years. His new headquarters are at the Roebling San Francisco district offices.

LOOKING AHEAD

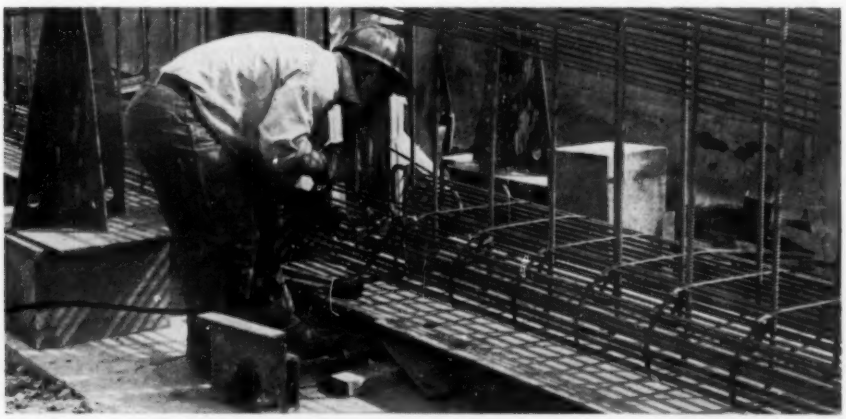
Sept. 15, 1958—6th International Congress on Large Dams, New York, N. Y.

Sept. 21-25, 1958—Prestressed Concrete Institute, Fourth Annual Convention, Edgewater Beach Hotel, Chicago, Ill.

Oct. 27-29, 1958—American Concrete Institute, Regional Meeting, Statler Hotel, Detroit, Mich.

Nov. 18-20, 1958—American Standards Association, Ninth National Conference on Standards, Hotel Roosevelt, New York, N. Y.

Feb. 23-27, 1959—American Concrete Institute, 55th Annual Convention, Statler-Hilton Hotel, Los Angeles, Calif.



APPLICATION OF STEEL STRAPPING to replace hoops of reinforcing rod in the manufacturing of prestressed concrete beams has been developed. The strapping is applied after the tensioning strands have been run out and is best applied while the strand is under tension. Passing the flexible steel strap around a bundle of strand involves no risk of snagging, kinking, or nicking. Workman here engaged in the strapping operation can use single tool for tensioning, sealing, and cutting the strap. Even when working under adverse conditions, one man can apply tension and seal at least 20 loops of strap in an hour, including his time moving from point to point along the casting bed. Under more favorable conditions he could apply up to 60 loops per hour. According to the Signode Steel Strapping Co., Chicago, a typical loop of strap together with the necessary sealing material will cost less than six cents

Honor Roll

January 1—May, 1958

Blas Lamberti is still in first place on the Honor Roll and he has raised his credits to 14. Joseph J. Waddell is in second place with 10 credits and Phil M. Ferguson is third with 8 credits.

The ACI Honor Roll emphasizes your continuing interest in Institute work by the promotion of new members. Each month, the Honor Roll lists the individuals who have extended their interest in ACI to their associates by inviting them to join the American Concrete Institute. Through the Honor Roll, ACI recognizes the efforts of each individual in furthering the influence of ACI. Will your name top the list next month?

Blas Lamberti	14
Joseph J. Waddell	10
Phil M. Ferguson	8
Ernst Maag	6
C. P. Siess	5
Ernest L. Spencer	5
Celso A. Carbonell	4
L. M. Legatski	4
J. P. Thompson	4
P. E. Ellen	3
Paul A. Hansen	3
Samuel Hobbs	3
Kenneth M. Huber	3
Frank E. Legg, Jr.	3
H. P. Mittet	3
Jerome M. Raphael	3
E. Schmid	3
Clarence A. Walkwitz	3
Byron P. Weintz	3
William M. Avery	2
Luther E. Bell	2
O. R. Bell	2
Moises Bendahan	2
Vincent R. Cartelli	2
Miles N. Clair	2
W. L. Collier	2
W. S. Cottingham	2
Jaime de las Casas	2
Aleck S. Evans	2
Martin J. Gutzwiller	2
Arch Hull	2
Thomas C. Kavanagh	2
F. R. Khan	2
R. G. King	2
Carl O. Knop	2
E. Vernon Konkel	2
Carl H. Koontz	2
George Kurio	2
James R. Libby	2
Ciceron Hiedras Lopez	2
William McGuire	2
Douglas McHenry	2
George A. Mansfield	2
Bryant Mather	2
Frank B. May	2
James A. Murray	2
Wendell H. Nedderman	2
H. C. Pfannkuche	2
Walter H. Price	2
Abdur Rahman S. Rasul	2
Clarence Rawhauser	2
Francesco Sardella	2
C. H. Scholer	2
George Shervington	2
Howard Simpson	2
Donald L. Strange-Boston	2
T. W. Thomas	2
Ellis S. Vieser	2
Jose Antonio Vila	2
Ivan A. Villamil	2
Tsu-ming Yang	2

New Members

The Board of Direction approved 91 individual applications, 4 Corporations, 8 Juniors, and 16 Students, making a total of 119 new members. Considering losses due to resignations, nonpayment of dues and deaths, the total membership on May 1, 1958 was 9454.

Individual

ADAMS, RICHARD S., Anchorage, Alaska (Cons. Engr. Arctic Alaska Testing Labs.)

ALONSO, MARIO LENS, Habana, Cuba (C. E. & Arch.)

ARAJS, ULDIS, Regina, Sask., Canada (Draftsman, Stock, Ramsay & Assoc.)

ARMOUR, CHARLES R., Atlanta, Ga. (Engr., Harry G. Hunter)

ARMSTRONG, ARNOLD H., Laprairie, Que., Canada (Inspector, St. Lawrence Seaway Authority)

AUSTILL, COL. HURIEOSCO, Mobile, Ala. (Cons. C. E.)

BAHERI, JAVAD, Mechanicsburg, Pa. (Bridge Designer, Dept. of Hwys., Harrisburg, Pa.)

BECK, J. S. R., Dundas, Ont., Canada (Mgr., Bldg. Div., C. C. Parker & Assocs., Ltd.)

BOEHMIG, ROBERT L., Jonesboro, Ga. (Pres., Morris, Boehmig & Tindel, Inc.)

BONNER, LANE L., JR., Charleston, S. C. (Sales Dept., Polychemicals Div., West Virginia Pulp & Paper Co.)

DRANICHAK, M., Tonawanda, N. Y. (Linde Co., Div. of Union Carbide Corp.)

BRICKETT, EDWARD M., St. Paul, Minn. (Pres., Construction Chemicals, Inc.)

BROCK, GEOFFREY, Birmingham, England (Lecturer in Civil Engrg., The University)

CARDAZZI, ETTORE, Los Caobos, Caracas, Venezuela (C. E.)

CASTORINA, LOUIS L., Oceanside, N. Y. (Reinforced Concrete)

COLBY, RALPH S., Merrimac, Mass. (Test Engr., Avco Mfg. Corp., Research & Advanced Dev. Div.)

COLEMAN, R. C., Fargo, N. D. (C. E. in chge. of Fargo Office, Kirkham, Michael & Assoc.)

COOPER, DONALD S., Lafayette, Ind. (Struct. Engr., Walter Scholer & Assoc.)

COOSE, HAROLD E., St. Louis, Mo. (Head Engr., Struct. Section, Bridge Div., Sverdrup & Parcel, Inc.)

CUNNINGHAM, HENRY JOSEPH, San Francisco, Calif. (Chf. Struct. Engr., Thomas B. Bourne Assoc.)

DARVAS, ROBERT M., Chicago, Ill. (Civil & Struct. Engr., Skidmore, Owings & Merrill)

DE PAOLI, RODOLFO, Montreal, P. Q., Canada (Struct. Engr., Truscon Steel Co. of Canada, Ltd.)

DUNCAN, ROBERT A., New York, N. Y. (Tech. Repres., North American Cement Corp.)

ELLIS, PAUL, University Heights, Ohio (Gen. Contr.)

ERLAND, MELVIN A., Portland, Ore. (Pres., Gen. Mgt., Ready-mix Concrete Co.)

EROSKEY, FRANK, Cleveland, Ohio (Owner, Frank Eroskey & Assocs., Struct. Engrg., Cons. Engr.)

FERRIS, ROBERT C., Buffalo, N. Y. (Field Engr., Erdman, Anthony & Hosley)
FONTAINE, RICHARD E., Worcester, Mass. (Asst. Chf. Draftsman, Morgan Constr. Co.)
GALVEZ, PEDRO R., Miraflores, Lima, Peru (C. E., Corporacion Peruana del Santa)
GNAEDINGER, JOHN P., Chicago, Ill. (Pres., Soil Testing Services, Inc.)
GRAY, J. ASHTON, Leesburg, Fla. (Mgr. & Techn. Concrete design, control, placing & inspection, Dura-Stress, Inc.)
GREENBANK, EARL E., Oregon City, Ore. (Ebasco Services, Inc.)
GROSSMAN, PETER, Vancouver, B. C., Canada (Director, Vancouver Public Library)
GUILLERMETY, GEORGE M., Ann Arbor, Mich. (Grad. Stud. in C. E., U of M)
HARVIE, FRANK B., Downey, Calif. (Design Engrg. Supv., Fluor Corp.)
HOLMAN, IRVIN C., Fargo, N. D. (Struct. Engr., Kurke Assocs.)
HOLMAN, RALPH M., Pasco, Wash. (Field Mng. of Western Projects, Holman Erection Co.)
HOSAKA, ROBERT M., Honolulu, Hawaii (C. E., Maint. Div., DPWO)
HUHTA, RICHARD S., Chicago, Ill. (Mng. Editor, Concrete Products Magazine, Maclean-Hunter Publishing Corp.)
ILGENFRITZ, J. J., Washington, D. C. (Dept. of Licenses & Inspections, D. C.)
IMAM AHMAD, S. A., Dacca, East Pakistan (Exec. Engr., Sole Chge of Hwy Bridge Design, Gov't of East Pakistan, Dept. of Communications & Bldgs.)
IVERSON, DONALD E., Madison, Wis. (Arch., Law, Law, Potter & Nystrom)
JENDROSZ, BERNARD KAROL, Ottawa, Ont., Canada (Cons. Engr., Dept. of National Defence)
JESSEN, R. A., Chicago, Ill. (Div. Mgr., Sika Chemical Corp.)
JOHNIS, STEVE, Lowell, Mass. (Struct. Engr., Metcalf & Eddy)
KELLY, ERWIN C., Cincinnati, Ohio (C & Struct. Engr., Hixson Tarter & Assoc.)
KLEIN, A., EEFDE (G) Netherland (Cons. Engr., Delft Techn Un.)
KLEIN, ALFRED H., Griffith, Ind. (C. E., Stauffer Chemical Co.)
KNARR, ROBERT E., Corpus Christi, Tex. (Tech. Dir., Halliburton Portland Cement Co.)
KRAUSSER, J. W., Cincinnati, Ohio (Assoc. Dir. of Constr. Dept., Engrg. Div., The Proctor & Gamble Co.)
LEWKOWICZ, JANUSZ JOZEF, Accra, Ghana (Sr. Design Engr., Gammon Ghana Ltd.)
MALLON, ARTHUR H., Milton, Mass. (Asst. Prof. of Civil Engrg., Merrimack College)
MARTIN, CHARLES F., JR., Fargo, N. D. (Engr., Civil, Testing, Constr. Line)
MCGRATH, JOSEPH K., Arlington Heights, Ill. (Chf. Struct. Engr., Kraft Foods)
MEANS, JACK A., Reno, Nevada (Cons. Engr., Struct.)
MELA, LUIGI, Caracas, Venezuela (C. E.)
MIDGLEY, WESLEY R., Monroeville, Pa. (Engr., U. S. Steel Applied Research Lab.)
NOYES, HAYDON T., New York, N. Y. (Chf. Engr., Hd. of N. Y. Engrg. Dept., Turner Constr. Co.)
OGAWA, TOSHI, Oakland, Calif. (C. E., Haluk Akol, Struct. Engr.)
OSTAPENKO, ALEXIS, Bethlehem, Pa. (Asst. Prof. of Civil Engrg., Fritz Lab., Lehigh Univ.)
PIEPER, DR. ING. KLAUS, Lubeck, Germany (Baurat, Mgr. of Bauverwallung der Stadl Lubeck)
PORTFIELD, JOSEPH, Los Angeles, Calif. (Cons. Engr.)
REICHARD, THOMAS W., Washington, D. C. (Lab. Testing, Physicist, NBS)
RETZLAFF, WILLIAM A., Milwaukee, Wis. (Sec.-Treas., Mgr., State Sand & Gravel Co.
ROSNER, ARNOLD S., Youngstown, Ohio (Cons. Engr.)
RUSSE, C. F., San Francisco, Calif. (Distr. Mgr., Pittsburgh Testing Lab.)
SABI, MARIO, Vibora, Havana, Cuba (C. E.)
SARAPU, FELIX RICHARD, Lane Cove, NSW, Australia (Sr. Design Engr., Woolacott, Hale & Bond)
SAVRAN, MANUEL, Linden, N. J. (Design Checker & Supv., Ramseyer & Miller)
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SELLERS, J. B., Mobile, Ala. (Asst. Supv. of Bridges & Bldgs., Louisville & Nashville R. R. Co.)
SINGH, BALDEO GOORIE, San Fernando, Trinidad, West Indies (Lab. Testing, Matls. & Research Engrg., Dept. of Works & Hydraulics)
SOKAL, ERNEST, Baltimore, Md. (Engr., Capitol Steel Corp.)
STIRBENS, VINCENT R., Youngstown, Ohio (Sr. Draftsman, Chge., Struct. Engrg., Copperweld Steel Co.)
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SWEITZER, R. J., Wharton, N. J. (Dir. of Research & Devel., Lock Joint Pipe Co.)
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VOGT, OTTO, Silver Spring, Md. (Reg'd Struct. Engr.)
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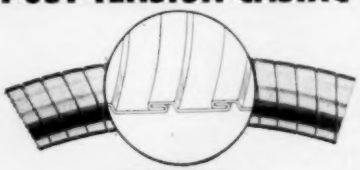

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COOPER, JONATHAN A., Hamilton, New Zealand (Asst. C. E., Hamilton City Council)
GUILFORD, ALBERT A., Angola, Ind. (Instructor, R/C Design & Civil Engrg. course, Tri-State College)
KLYMAN, JOSEPH C., Eastport, Md. (Design & Drafting, Struct. Dept., Whitman & Requardt)
KNEUER, PAUL R., Allentown, Pa. (Service Engr., Lehigh Portland Cement Co.)
PUMAREJO, JAIME, Barranquilla, Colombia (Civil Engrg., Assoc. Member Mgr., Inar Ltda.)
RAMIREZ, JOSE ANTONIO, Sabana Grande, Puerto Rico (C. E.)

Student

AGUILAR, RODOLFO J., Baton Rouge, La. (La. State Univ.)
BAHOOR, ARSHAD, Baghdad, Iraq (Higher Industrial Inst.)
COHEN, JACQUES, Champaign, Ill. (Univ. of Ill.)
CROUCH, CHARLES L., Grand Prairie, Tex. (Southern Methodist Univ.)
DONNELLY, MICHAEL ALAN, Reading, Mass. (Univ. of Maine)
ESTRELLA, ALVARO, Ann Arbor, Mich. (U. of M.)
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HADLEY, JAMES ALEXANDER, Christchurch, New Zealand (Univ. of Canterbury)
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POCASANGRE, SAUL MAXIMINO, Ann Arbor, Mich. (U of M)
SKOK, EUGENE L., JR., St. Paul, Minn. (Univ. of Minn.)
TAYLOR, VESPER, Decatur, Ill.
TERAN P., EDUARDO A. MIER Y, Caracas, Venezuela
WILSHAW, RONALD ENOCH, Detroit, Mich. (Wayne State Univ.)

Errata

The following correction should be made in "Concept of Elastic Parameters," by Valerian Leontovich, in the May 1958 JOURNAL.

p. 995—Line 2 should read in part ". . . reactions at the left and right ends of the member . . ."

* * *

The following corrections should be made in "Transverse Strength of Concrete Block Walls," by F. W. Cox and J. L. Ennenga, in the May 1958 JOURNAL.

p. 957—Line 18, delete "and 7."
—Line 20, change 650 to 649.

p. 958—Table 2, add "Maximum span" to heading of third column.

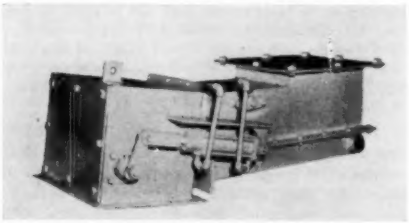
p. 960—Under "conclusions," seventh line should read in part ". . . the B and D walls contained joint reinforcement . . ."

Tools, Materials, Services

Under this heading note is made of producer literature and products of presumed technical interest to ACI users of tools, equipment, materials, accessories, and special services.

Air-powered cement feeder

The Butler Airflomatic cement feeder, according to its manufacturer, has no moving parts, no pins or bolts to shear, and no clutches to break, wear, or slip. The



feeder employs a cushion of high volume, low pressure air on which the cement rides; the current of air is supplied by a blower which also aerates and fluffs the cement in the overhead bin. No jets for additional air are required. It is further stated that the feeder is quick and easy to install and can be used with all road batching or ready-mixed concrete plants regardless of make.—Butler Bin Co., Waukesha, Wis.

Soiltest inaugurates quarterly news letter

To further disseminate information regarding equipment and progress in testing of soils, concrete, and bituminous materials, Soiltest, Inc., has instituted a 4-page news letter, *The Testing World*, which will be issued quarterly. The publisher will add names to the complimentary mailing list on request.—Soiltest, Inc., 4711 W. North Ave., Chicago 39, Ill.

Marble-like precast stone

KoverStone is a "nonporous" square cut stone with a natural stone-like surface. The manufacturer states that the cast stone, developed after 35 years of research, is free from surface holes, thus preventing the danger of water freezing and expanding within the stone, thereby eliminating one of the major causes of cracking. It is available now in a marbleized white, and blends of buff, brown, and cocoa color.—U. S. Aluminum Siding Corp., 10551 West Anderson Place, Franklin Park, Ill.

Fastening device for plywood forms

A fastening device for plywood concrete forms holds both the form tie and waler in place. Manufacturer reports the new system speeds forming 20 to 40 percent. The bracket, called a Ply-Tie holder, is designed for use with conventional fir plywood panels. One bracket arm has a tear-drop slot which fits over the head of a special form tie. The other arm holds the waler.

The tie holes are predrilled in the plywood, using a standardized pattern. Then, the contractor tacks 2 x 4 studs to the plywood with one of the studs backing the joint, and the holder secures the form tie and holds the waler in place according to the manufacturer. The same holder can be used for light, medium, or heavy construction by varying the spacing and thickness of studs and walers.—Trueforms, Inc., 414 Times Square Bldg., Seattle 1, Wash.

Portland cement facts

Walter J. McCoy, director of research for the Lehigh Portland Cement Co., has prepared a 15-page booklet based on addresses recently made before a number of concrete conferences.

The title "The ABC's of Portland Cement" was chosen as a convenient way of indicating the three general parts of this booklet. The first part is a short review of the history and development of portland cement and concrete; Part B is a discussion of the raw materials and manufacturing process; Part C explains explanation of the chemistry of the finished product and the basic types of portland cement. Booklet copies are available on request.—Lehigh Portland Cement Co., Allentown, Pa.

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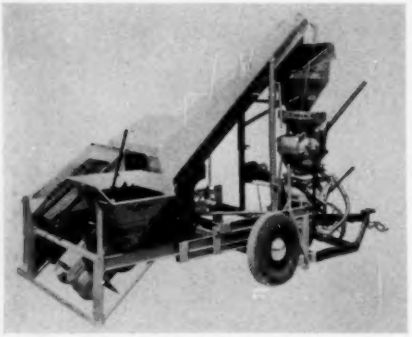
Mix-elevator units

Model 600 and Model 610, Airplaco Mix-Elevators, designed for high production and continuous aggregate proportioning, mixing, elevating, and screening for all concrete guns, are basically the same except that the



Model 610 incorporates a built-in aggregate dryer, which makes it possible to reduce the moisture content of the sand by 2 to 5 percent. With this reduction in moisture content, the manufacturer reports, the gunning operation can be continued in wet weather or when the moisture content of the sand is too high to permit a satisfactory operation otherwise.

The drying apparatus includes a 750,000 Btu propane or LP gas burner, with the necessary duct work directing the hot air blast into the sand bin and along the length of the mixing trough. Both models can be operated at any desired rate up to 12 cu yd per hr.—Air Placement Equipment Co., 1009 W. 24th St., Kansas City, Mo.

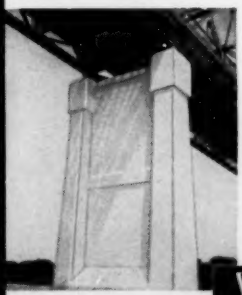
Aeration system for blending dry pulverized materials in bulk

An innovation in the blending of dry bulk pulverized materials which reduces deviations in unblended components down to ± 0.2 percent was recently announced. Known as "Fuller Airmerge Quadrant Blending," it is basically an aeration system and can be used on any materials which can be fluidized by compressed air; these would include cement, fly ash, and materials with similar physical characteristics. The system is suited for use in either batch or continuous schemes of operation. —Fuller Co., Catasauqua, Pa.

Slurry pump

Bulletin 185 describes capacities and applications of heavy-duty, low-speed RX slurry pump for cement, sand, coal, and other solids. Operating data and specifications for continuous duty heavy, fine, or coarse slurries are given. Pump is designed to handle abrasives up to maximum fluid consistency under suction lift or positive head.—Morris Machine Works, Baldwinsville, N. Y.

WESTERN
WATERPROOFING COMPANY
restores
concrete piers of
MacArthur Bridge,
Burlington, Iowa



When the concrete bridge piers began to disintegrate so badly that reinforcing was exposed, Western was called on to do the repair job. The piers were restored with gun-applied concrete and weather-proofed with special water-repellent grout. The caps, in exceptionally poor condition, were completely rebuilt.

Prevent serious structural weakness and huge repair bills on any concrete structure . . . consult Western. All work done under contract, fully insured and guaranteed. No materials for sale. *T. M. Reg.

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Literature Available

Pertinent details on the latest equipment and products on the market are available in recently released literature. Exact titles of the booklets and catalogs are indicated in capital letters. They may be requested directly from the manufacturers listed below.

GRAVEL BENEFICATION PLANTS—an illustrated booklet describes the removal of soft stones from aggregate, explains the theory of the Blaw-Knox benefication process for economical production of high grade aggregate from local gravel deposits.—Blaw-Knox Co., Construction Equipment Division, Mattoon, Ill.

HERE IT IS—folder describes recently introduced surfacing material Penntrowel for corrosion protection of concrete. Shows how, when, and where to use.—Pennsalt Chemicals Corp., 3 Penn Center, Philadelphia, Pa.

THE PRESCON SYSTEM—covers history and development of concrete prestressing, particularly post-tensioning by the Prescon method. Guide specifications and detail drawings are included.—The Prescon Corp., Box 4186, Corpus Christi, Tex.

PHILLIPS RED HEAD BUYERS GUIDE, No. 57—Illustrated catalog devoted to concrete fasteners includes prices, test reports, specification data, and comparative charts.—Phillips Drill Co., Michigan City, Ind.

RIMAS CONCRETE PIPE VIBRATING MACHINE, TYPE 602 and RIMAS CONCRETE PIPE VIBRATING MACHINE TYPE 1001—Two separate folders each giving complete description of the respective models, state production scope, production capacity, and advantages of each unit. Type 602 is suitable for manufacturing concrete pipes in diameters from 10 to 60 cm; Type 1001 is designed for manufacturing concrete pipes from 10 to 120 cm.—Ringsted Jernstoberi and Maskinfabrik, Ltd., Ringsted, Denmark

STRESSTEEL POST-TENSIONING—Catalog No. SS-3, 23-pages, pictures typical uses of Stressteel in bridges, building frames, cast-in-place structural members, pavements, dams and retaining walls, prestressed tanks, and strengthening existing structures. Shows components of Stressteel tensioning units, specifications, and recommended procedures.—Stressteel Corp., 221 Conynham Ave., Wilkes-Barre, Pa.

CONCRETE PRESSURE PIPE LAYING INSTRUCTIONS—revised edition of this pocket size manual, in 16 pages boils technical data down to practical application. On-the-job photos with brief explanations illustrate step-by-step procedure in concrete pipe laying; tables of pipe sizes and deflection data are included.—Price Brothers Co., 1932 East Monument Ave., Dayton, Ohio

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Dockstader retires after 40 years with Stone and Webster

After more than 40 years active engineering practice with Stone and Webster Engineering Corp., Boston, Ernest A. Dockstader recently announced his retirement. A member of ACI since 1922, Mr. Dockstader has been active in Institute affairs for many years. In 1925 he was awarded the Wason Medal for his paper, "Report of Tests Made to Determine the Temperatures in Reinforced-Concrete Chimney Shells," judged to be the most meritorious paper of the year.

He served as chairman of ACI Committee 505 for several years while the committee was in the process of studying and revising an existing standard, ultimately resulting in the adoption in 1954 of "Specification for the Design and Construction of Reinforced Concrete Chimneys (ACI 505-54)." Mr. Dockstader is currently serving as ACI representative to the American Standards Association, Sectional Committee A52, Building Code.

Expansion joint group elects officers

Wallace C. Fischer, president of Servicised Products Corp., Chicago, was elected president of the Expansion Joint Institute at its recent annual meeting. William E. Hagemeister, sales manager of the construction materials division of Presstite-Keystone Engineering Products Co., a division of American-Marietta Co., St. Louis, was elected vice-president of the group. H. G. Meadows, president of W. R. Meadows, Inc., Elgin, Ill., was elected treasurer.

F. W. Lagerquist, merchandise manager, roofing and allied products department of the Celotex Corp., Chicago, a past president of the Expansion Joint Institute, was elected to the board of directors as were the incoming officers.

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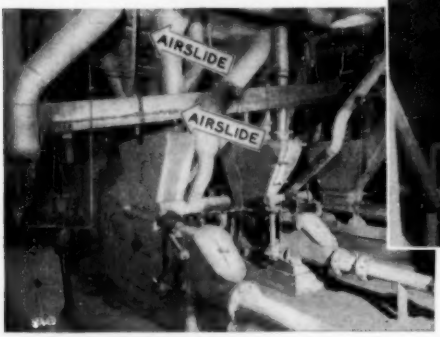

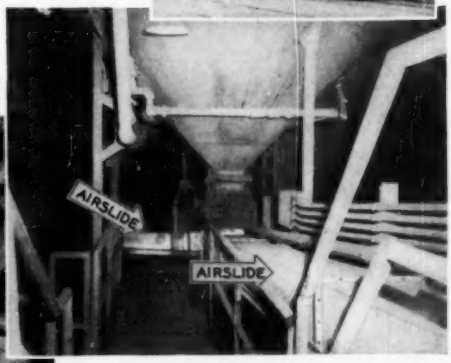

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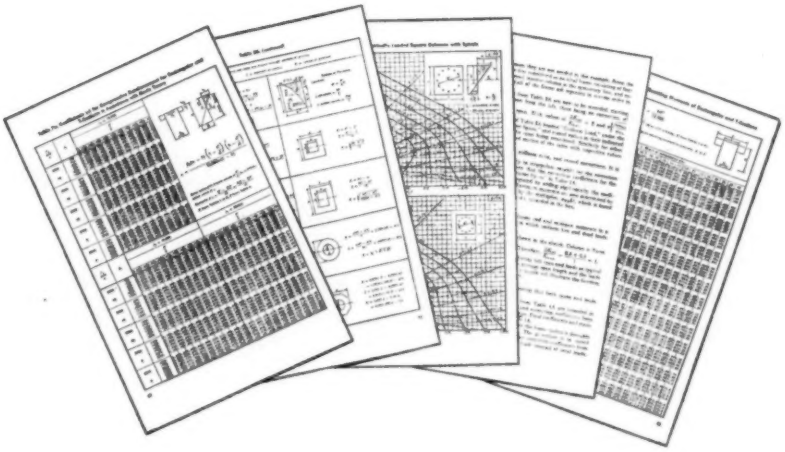

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DISCUSSION, PROCEEDINGS V. 54

Discussion of papers published in the October through December, 1957 JOURNALS appears in the concluding pages of this June issue, as the Institute continues its quarterly publication of discussion. Papers published July through September, 1957, were discussed in the March, 1958 issue; discussion of papers published January through March, 1958 will be published in Part 2, September, 1958 ACI JOURNAL. April through June discussion will appear in Part 2, December, 1958 along with index and errata for V. 54.

Listed below are the papers whose discussion is published in this JOURNAL.

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